Class T 620.6
Book No. 31417

Northeastern University
Library

John R. Moses

TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XXX.

OCTOBER TO DECEMBER, 1893.

NEW YORK:

PUBLISHED BY THE SOCIETY.

1893.

Entered according to Act of Congress, by the AMERICAN SOCIETY OF CIVIL ENGINEERS, in the Office of the Librarian of Congress, at Washington.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CONTENTS.

PAPERS.

NO.	MONTH.		PAGE
633	*October.*	The Testing of Portland Cement and the Development of the Cement Industry in Germany. MAX GARY	1
634	*do.*	On the Manufacture and Testing of Portland Cement. HENRY FAIJA	43
635	*do.*	Topographic Surveys. HERBERT G. OGDEN	62
636	*do.*	Recent Experience on the U. S. Coast and Geodetic Survey in the Use of Long Steel Tapes for Measuring Base Lines. R. S. WOODWARD	81
637	*do.*	Terrestrial Magnetism in North America. CHARLES A. SCHOTT	108
638	*do.*	Fundamental Units of Measure. T. C. MENDENHALL	120
639	*do.*	Historical Notes upon Ancient and Modern Surveying and Surveying Instruments. H. D. HOSKOLD	135
640	*do.*	The Treatment of Metals for Structural Purposes. JAMES CHRISTIE	155
641	*do.*	A Proposed Method of Testing Structural Steel. ALFRED E. HUNT	181
642	*do.*	The Use of Mild Steel for Engineering Structures. GEORGE C. MEHRTENS	204
643	*do.*	The Use of Basic Mild Steel as Material for Construction in Germany. C. WEYRICH	219
644	*November.*	The Plant of Maritime Commercial Ports of France. H. DESPRES	235
645	*do.*	The Development of Quay Cranes in the Port of Hamburg. CHR. NEHLS	258
646	*do.*	Method used to Secure the Stability of a Quay Wall at the Port of Altona, on the Elbe, in Germany, which had Shifted its Position after Completion. BERTHOLD STAHL	284
647	*do.*	Experiences had During the Last Twenty-five Years with Water Works having an Underground Source of Supply. B. SALBACH	293
648	*do.*	Some Questions Concerning the Filtration of Water. W. KÜMMEL	330
649	*do.*	On the Distribution of Water in the City of Mexico. L. SALAZAR	336
650	*do.*	Purification of Sewage and of Water by Filtration. HIRAM F. MILLS	350
651	*do.*	The Sewerage System of Milwaukee and the Milwaukee River Flushing Works. G. H. BENZENBERG	367

31417

NO.	MONTH.		PAGE
652	*November.*	The Enlargement and Improvement of the North Sea Canal of Holland (Amsterdam Ship Canal).	
		A. E. KEMPEES	386
653	*do.*	Notes on Projects for the North Sea Canal from 1629–1893.	
		A. HUET	416
654	*do.*	The North and East Sea Canal.	
		J. FÜLSCHER	420
655	*do.*	The New Railway Stations at Dresden.	
		C. KÖPCKE and O. KLETTE	450
656	*December.*	Inland Transportation.—Discussion on Paper No. 605.	
		By C. W. RAYMOND	457
		EDW. P. NORTH	467
		F. A. MAHAN	474
657	*do.*	Subaqueous Rock Excavation.—Discussion on Paper No. 606.	
		By O. M. POE	478
		JAMES DEAS	482
658	*do.*	Improvement of Rivers.—Discussion on Paper No. 607.	
		By TH. REHBOCK	483
		C. O. GLEIM	485
		WM. MURRAY BLACK	486
		W. R. HUTTON	487
659	*do.*	Harbor Improvement.—Discussion on Paper No. 608.	
		By JOHN PURSER GRIFFITH	490
660	*do.*	Improvement of Rivers.—Discussion on Paper No. 609.	
		By H. ENGELS	492
		WM. MURRAY BLACK	493
		W. R. KING	494
		JOHN PURSER GRIFFITH	496
661	*do*	Improvement of Harbors —Discussion on Paper No. 610	
		By JOHN PURSER GRIFFITH	498
		WM. MURRAY BLACK	500–509
		WM. P. CRAIGHILL	503
		THOMAS W. SYMONS	506
662	*do.*	Railway Terminal Systems.—Discussion on Paper No. 612.	
		By C. O. GLEIM	512
663	*do.*	Loadings for Railway Bridges.—Discussion on Paper No. 618.	
		By WARD BALDWIN	515
		J. A. L. WADDELL	522
		C. D. PURDON	523
664	*do.*	Surveys for Railway Location.—Discussion on Paper No. 619.	
		By GEORGE R. HARDY	525
		G. B. NICHOLSON	526
		C. O. GLEIM	526
		ARTHUR PEW	526
		A. BRYSON	528
		W. R. KING	530
		F. A. GELBCKE	530
665	*do.*	Calculation of Cross Sections.—Discussion on Paper No. 620.	
		By ARTHUR PEW	533
		FRANCISCO DA SILVA RIBEIRO	534
666	*do*	The Gauges of Railway Track.—Discussion on Paper No. 621.	
		By H. G. PROUT	537
		FOSTER CROWELL	538
		E. E. R. TRATMAN	541
		E. A. ZIFFER	541
667	*do.*	Railway Signaling.—Discussion on Paper No. 622.	
		By GEORGE R. HARDY	546

NO.	MONTH.		PAGE
668	*December.*	Transmission of Power in Operating Cable Railways.—Discussion on Paper No. 623.	
		By A. BRYSON	548
		DANIEL BONTECOU	550
		ROBERT GILLHAM	554
669	*do.*	The Continuous Superstructure of the Memphis Bridge.—Discussion on Paper No. 624.	
		By GEORGE H. THOMSON	559
		HENRY GOLDMARK	560
		Sir BENJAMIN BAKER	561
		ROBERT MOORE	562
		CHARLES MACDONALD	563
		GEORGE S. MORISON	564
		ALFRED D. OTTEWELL	565
670	*do.*	Bridge Substructure and Foundations in Nova Scotia.—Discussion on Paper No. 625.	
		By K. SUTHERLAND	567
		F. W. W. DOANE	569
		W. R. BUTLER	571
		E. GILPIN	573
		P. S. ARCHIBALD	574
		H. S. POOLE	576
		MARTIN MURPHY	576
671	*do.*	Subaqueous Foundations.—Discussion on Paper No. 626.	
		By C. O. GLEIM	579
		F. NEUKIRCH	579
672.	*do.*	American Grain Elevators.—Discussion on Paper No. 627.	
		By EDWARD B. TAYLOR	583
		E. LEE HEIDENREICH	584
673	*do.*	Paving Brick.—Discussion on Paper No. 628.	
		By DANIEL W. MEAD	585
		WILLARD BEAHAN	586
		F. A. CALKINS	586
		ALBERT SMITH	588
674	*do.*	Principles for the Laying Out of Cities.—Discussion on Paper No. 632.	
		By JAMES OWEN	591
675	*do.*	Manufacture and Testing of Portland Cement.—Discussion on Papers Nos. 633 and 634.	
		By D. J. WHITTEMORE	594
		R. W. LESLEY	594
		WM. MURRAY BLACK	601
		Sir BENJAMIN BAKER	603
		WALTER R. KINIPPLE	604
		JOHN P. GRIFFITH	605
		HENRY FAIJA	607
676	*do.*	Topographic Surveys.—Discussion on Paper No. 635.	
		By CHARLES B. BRUSH	611
		KENNETH ALLEN	612
		HENRY GANNETT	615
		MARCUS BAKER	619
		HERBERT G. OGDEN	632
677	*do.*	The Use of Long Steel Tapes —Discussion on Paper No. 636.	
		By ROBERT MOORE	638
		CHARLES B. BRUSH	639
		J. A. OCKERSON	639
		O. B. WHEELER	641
		H. M. MARSHALL	642

NO.	MONTH.		PAGE
678	*December.*	Terrestrial Magnetism in North America.—Discussion on Paper No. 637.	
		By HENRY GOLDMARK	653
		R. S. WOODWARD	653
		CHAS. A. SCHOTT	654
679	*do.*	Treatment of Metals for Structural Purposes.—Discussion on Paper No. 640.	
		By L. L. BUCK	655
		A. E. HUNT	657
		J. E. GREINER	659
680	*do.*	A Proposed Method of Testing Structural Steel.—Discussion on Paper No. 641.	
		By GEO. S. MORISON	666
		L. L. BUCK	667
		T. S. WHITE	668
		JAMES CHRISTIE	669
		A. C. CUNNINGHAM	669
		JOHN P. GRIFFITH	674
		ALFRED E. HUNT	675
681	*do*	The Use of Mild Steel.—Discussion on Paper No. 643.	
		By G. H. THOMSON	681
		D. J. WHITTEMORE	682
		GEO. S. MORISON	682
		ROBERT MOORE	683
		WILLIAM METCALF	684
		C. WEYRICH	685
682	*do.*	Stability of Quay Walls.—Discussion on Paper No. 646.	
		By PETER VEDEL	687
		C. O. GLEIM	689
683	*do.*	Underground Sources of Water Supply.—Discussion on Paper No. 647.	
		By CHARLES B. BRUSH	690
		L. L. TRIBUS	694
		D. J. WHITTEMORE	696
		D. W. MEAD	696
		B. SALBACH	701
684	*do.*	Purification of Sewage and of Water by Filtration.—Discussion on Paper No. 650.	
		By EDWARD FLAD	702
		ALLEN HAZEN	702
		RUDOLPH HERING	703
		THOS. M. DROWN	705
		G. H. BENZENBERG	706
		ROBERT MOORE	707
		E. C. JORDAN	708
685	*do.*	The Sewerage System of Milwaukee.—Discussion on Paper No. 651.	
		By D. J. WHITTEMORE	709
		F. P. STEARNS	710
		G. H. BENZENBERG	711
686	*do.*	The North Sea Canal of Holland.—Discussion on Paper No. 652.	
		By J. FÜLSCHER	712
		R. W. LESLEY	713
		THEODOR REHBOCK	713
		W. R. HUTTON	714
Engineering Congress.—Complete List of Papers			718

ILLUSTRATIONS.

PLATE.	MONTH.		PAPER	PAGE
I–II.	*October.*	Surveying Instruments	639	154
I.	*do.*	Basic Martin Steel	642	218
II.	*do.*	Thomas Mild Steel	642	218
I–IV.	*November*	Plant of French Ports	644	257
I–VI.	*do.*	Quay Cranes at Hamburg	645	282
I.	*do.*	Quay Wall at Altona, Germany	646	291
I–III.	*do.*	Sewerage System of Milwaukee	651	384
I–VIII.	*do.*	North Sea Canal of Holland	652	415
I–IX.	*do.*	North and East Sea Canal	654	449
I.	*do.*	Railway Stations at Dresden	655	456
I–II.	*December.*	Bridge Substructure in Nova Scotia	670	568
I–II.	*do.*	Topographic Surveys	676	613
I.	*do.*	Use of Long Steel Tapes	677	646

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

633.

(Vol. XXX.—October, 1893.)

THE TESTING OF PORTLAND CEMENT AND THE DEVELOPMENT OF THE CEMENT INDUSTRY IN GERMANY.

By MAX GARY, Civil Engineer, Berlin, Germany.

Translated from the German by JOHN S. SIEBERT, Instructor in Civil Engineering, Lehigh University.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

When, in 1852, the idea of utilizing the English invention of the manufacture of Portland cement first appeared in Germany, no one anticipated the growth this industry was destined to have, a growth which soon outstripped that of the mother-country, England.

As early as 1824, Joseph Aspdin, of Leeds, made a hydraulic lime by calcining a certain mixture of slaked lime and clay, which on account of its resemblance in color and hardness to the Portland stone, much valued at that time in England, he called Portland cement. The English product was soon introduced into Germany, and for a long time was considered unequaled. Not till 1852 was attention directed to the occurrence of the septaria clay in Pommerania, on the banks of the Oder, which was declared by Dr. Hermann Bleibtreu, of Bonn, to be

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

suitable material for the manufacture of cement. He also, in conjunction with Consul Gutike, erected at Züllchow, near Stettin, a small plant for experimental purposes, which led in 1855 to the founding of the first Stettin Portland cement factory (Lossius & Dellbrück), on what is now the property of the Züllchow works. Here, for the first time in Germany, Portland cement was made out of Stettin clay and chalk from the Island of Wollin, a cement which equaled the English product in quality and so quickly found favor that the annual production of 30 000 barrels was sold without special difficulties.

This success incited imitation, and in rapid succession the factories (most of them in existence to-day) of Bonn, "Lebbin, Oppeln," Lüneburg, Amöneburg, Finkenwalde and others sprang into existence. In 1877, there were in Germany 30 Portland cement factories, which at first copied closely the English methods of manufacture. At present the method of manufacture varies greatly in the different factories, according to the combination in which the requisite ingredients, carbonate of lime and clay, occur in the limestone, marble, chalk and other raw materials used.

Factories following the English methods generally employ the wet process.

The selection of the clays when a pure septaria cannot be employed, and the mixing in proper proportions of lime and clay (about 75% carbonate of lime to 25% of clay), constitute the real difficulties in cement-making to-day. After this the principal requirement for obtaining good Portland cement lies in a thorough mixing of the ingredients which are to be chemically combined by calcination. The two other divisions of manufacture, calcining at a very high temperature (burning to slag), and the grinding of kiln products, are well understood. Unquestionably, the wet process of to-day does not mix the ingredients so efficiently as the dry process, especially in those factories which work a fine limestone and mortar; but it is not to be denied that the rapid improvements in mill machinery may again cause a change in this. In some factories a combination of the two methods has already been adopted, the so-called half-dry process being used, in which the fine mortar and clay in proper proportions are mixed partly dry, in order to obtain the benefits of the best parts of each process. But mixtures of limestone and clay which may be at once burned to cement are rarely found in Nature.

It would exceed the limits of this paper to enter into a description of the chemical qualities of Portland cement and the development of the methods of manufacture, the making, drying, burning and grinding of the cement bricks; let us rather deal with the finished product and its mechanical properties, and state in what manner the quality of German Portland cement was systematically improved and how, in Germany more than elsewhere, efforts have been made, with the aid of scientific investigations, to probe the nature of Portland cement, bring its hidden properties to light and render them useful.

Before German Portland cement gained the world-wide fame it now enjoys, it had to overcome manifold difficulties. From the first, the competition with the English factories was severe. The low shipping rates and the extensive commerce of the home of Portland cement constantly drew large quantities of the English product to the German ports, and the reduction made in railroad tariffs for the benefit of these ports favored the introduction of a large part of this product into the interior. At the large ports on the North Sea the effect of English competition was especially marked, owing to the comparatively great distance of the German cement factories, and to this day the English product has not been entirely displaced. But in Central and South Germany the English competition, though favored by the natural waterway of the Rhine, has been compelled to abandon the field in favor of home factories erected in that region. Prejudice against German Portland cement and in favor of the foreign product was a further obstacle to its rapid introduction, to overcome which required long-continued efforts of the German manufacturers, perfected methods of manufacture and unceasing scientific labors of the large number of investigators who entered the service of this industry. To-day every German factory has in its employ one or more scientifically educated chemists whose duty it is to study the raw materials and the intermediate and finished products. Besides the powerfully equipped English factories, the competition of continental neighbors, especially Austria and Switzerland, had to be overcome; and, finally, the greatest of all obstacles, the prejudices against it arising from the long use of other low-priced and poorer mortar materials and ignorance of the new. The lack of knowledge regarding methods of test on the part of the consumer caused cheaper varieties of cement to be used and created a deep distrust of Portland cement, not only among builders, but even among the authorities.

Despite all these obstacles, the German Portland cement industry grew rapidly, as may be seen from the following table, based on the annual voluntary reports of the members of the Association of German Cement Manufacturers, who represent the major part of the German cement interests.

Year.	Number of factories.	Annual product in barrels @ 170 kg. (374 lbs.) net	Year.	Number of factories.	Annual product in barrels @ 170 kg. (374 lbs.) net.
1877.....	29	2 400 000	1887.....	45	7 050 000
1882.....	32	3 050 000	1888.....	52	7 950 000
1883.....	34	4 000 000	1889.....	60	8 800 000
1884.....	37	4 700 000	1890.....	60	9 150 000
1885.....	42	5 000 000	1891.....	60	9 950 000
1886.....	42	5 700 000	1892.....	62	10 600 000

More precise figures, giving a faithful picture of the development of the German cement industries during the last years, are available only since the introduction of the accident insurance law and the organization of industrial associations. According to the official reports of the quarry associations to which the German cement manufacturers belong, the development of this industry may be shown by the following statement:

Year.	Number of cement factories.	Number of men *permanently employed @ 300 working days.	Salaries paid. (Marks.)	Year.	Number of cement factories.	Number of men * permanently employed @ 300 working days.	Salaries paid. (Marks.)
1886....	124	11 883	8 263 437	1889....	149	16 388	12 294 665
1887....	121	13 610	9 555 614	1890....	159	19 174	14 839 656
1888....	146	13 352	11 108 539	1891....	158	19 503	15 166 627

* The number of "men permanently employed" was computed by dividing the total number of days all the men were employed in any one year by 300, this being taken as the customary number of working days in one year.

From the first table it is seen that the number of large Portland cement manufactories in Germany has been doubled in 15 years and the output increased fourfold in the same time.

From the second table it may be seen that the number of workmen employed and the sum paid in wages nearly doubled in five years. This table also includes a number of small factories, notably those of upper Bavaria, which employ but few hands and whose market is confined to

the country immediately adjoining. Such a tremendous growth of a comparatively young industry in so short a time was rendered possible only by the fact that the knowledge of the value of the Portland cement spread rapidly in the interior; that the demand from this region grew accordingly, and that the German product was successful in overcoming the opposition of foreign brands. The impartial recognition of the merits of German Portland cement by well-known foreign authorities, especially those of England, did much to further this end.

Mr. Henry Reid, author of "The Science and Art of the Manufacture of Portland Cement," in his book, "A Practical Treatise on Natural and Artificial Concrete,"* concludes his remarks about the difficulties met by cement manufacturers and builders in the manufacture and use of Portland cement, with the explanation that English manufacturers have every reason to improve their product. That this is not difficult, although necessary, he proves by the example set by the German Portland cement manufactory of "Stern," Toepffer, Grawitz & Co., at Stettin. He gives a description of their factory and what it had accomplished, showing that German Portland cement, even at that time, was able to compete with English cement, and promised eventually to become everywhere an opponent not to be left out of consideration.

What is here said of the one brand answers for all the German output of that time ; the German manufacturers were most earnestly engaged in securing recognition of their product throughout the world. As a result, General Gillmore, as President of the Committee of Award at the Centennial Exposition in Philadelphia, designated the "Stern" cement as the best Portland cement sent there ; and Professor Renleaux was enabled to announce at the session of the Central Association of Commercial Geography at Berlin, April 22d, 1880, that the reports of the Australian World's Exhibition held in Sydney declared: "The German cement was unquestionably the best." To-day this cement finds a market in all parts of the globe. The foreign countries grouped about the Baltic are regular customers, as also are Southern Russia, Switzerland, Belgium and Holland. But the sale in European countries probably falls short of that in the transoceanic ones. The United States of North America have thus far afforded the largest market. The countries of the Pacific Ocean come next, especially the more important harbors of Australia, as well as Japan and China.

* Published 1879, by E. & F. M. Spon, 46 Charing Cross, London.

Efforts for constant improvement went hand in hand with the growth of the German cement trade, not only because the field so laboriously won at home had to be maintained, but because it was necessary to widen the market for the increased production. To do this, it was absolutely necessary to understand the properties of cement, and how to utilize the same to full advantage. To reach these ends was the chief aim of the Association of German Cement Manufacturers, founded in 1877, which, with but few exceptions, all German manufacturers have joined, together with 18 Portland cement manufacturers of foreign countries. This association, at its general annual session, furnishes its members an opportunity to increase their knowledge of the properties of cement by verbal exchange of ideas, while the distribution of the printed stenographic reports of the session insures the publication of new facts and discoveries in the realm of cement, and enables the building public to keep pace with the advances in cement technique. Up to the time of the formation of this society uniform methods of judging the mechanical properties of cement were totally wanting, and testimony of the earnest work of the society may be found in the fact that, in 1877, in conjunction with the German Brick-Makers' Association, the Berlin Association of Architects, and the association "Berliner Baumarkt," succeeded in laying before the Royal Prussian Bureau of Commerce, Trade and Public Works a set of specifications, which were accepted by said bureau as "Standards for the Uniform Delivery and Testing of Portland Cement." These were issued to the building authorities on November 10th, 1878, for the Kingdom of Prussia, and were later adopted, also, by the remaining German States. From this time the history of the Association, under the guidance of its chairman, Dr. Dellbrück, is identified with the history of the development of the German Portland cement industry, to which development the German manufacturers of machinery and apparatus for Portland cement manufacture, headed by The Iron Works, formerly Nagel & Kämp, of Hamburg, have contributed no small part.

In this epoch of development belongs the first conference, afterwards called the Munich Conference, which was convened in the fall of 1884 by Professor Bauchinger, for the proper utilization of the results of tests made on various materials in different places. The first conference, which was attended by 79 representatives, from Germany,

Hungary, Russia and Switzerland, was soon followed by a second one, held in Dresden, in 1886, and the resolutions there adopted regarding uniform methods of investigation to be pursued in testing the mechanical properties of building and structural materials carried the knowledge of testing methods throughout the world, and awakened interest in those circles which had hitherto abstained from taking part in this matter.

On the strength of these resolutions a revision of the rules was completed in 1886, and these revised rules, after being submitted for opinion to the chief representative of the Royal Prussian Testing Station for Building Materials, Professor Dr. Böhme, as well as the Royal Academy of Architecture, and having been approved by these, with unimportant corrections, were officially promulgated on July 28th, 1887, by the Minister of Public Works for Prussia.

Without a doubt, Germany gained by these means the first rank, from the scientific standpoint of cement technology, among those countries which produce cement in any quantity, and its example has caused other countries to adopt similar action. In Austria, Switzerland and Russia additional rules were later added to the German ones, which at the time deviated in several particulars from the latter, but which at present agree with them on the whole.

In France, England, America, Australia and other countries where cement is largely used, there are no such rigid and comprehensive regulations as the German ones. Certain rules which have gained recognition through tradition and development in the course of time are considered sufficient. The third conference, held in the fall of 1890, in Berlin, which was attended by representatives from all countries, did much to bring the rules enforced in these countries into agreement. Meanwhile the German standards served as laws for the manufacturers, and produced further perfection of the manufactured article, which has been of overwhelming benefit to the public.

The fact that to-day all German factories which belong to the Association turn out a product which, in its principal qualities, never falls below a certain limit amply sufficient for the purposes of the material, is one but rarely found in the extensive manufacture of any article. Viewed from the consumers' standpoint, this is a highly welcome fact, insuring as it does, a guarantee of uniformity of the product at any place and at all times.

It must, however, be well borne in mind that the use of these regulations in no wise guarantees a uniform quality of the product of different factories, or that dependence should be placed upon them without previous tests; but, rather, that while a line of minimum quality has been drawn, no upper limit exists, and it is free to every factory to go as far above the minimum as the various circumstances in regard to location, quality of raw material, etc., permit.

In but one direction did the Association of German Cement Manufacturers deem it incumbent upon itself to go beyond the required regulations and give to the consumer a further, far-reaching guarantee. After the promulgation of the regulations, the practice of mixing different qualities of cement assumed such threatening proportions that the Association, in order to protect the good name of German Portland cement, and at the same time insure construction work against the use of unsound cement, found itself forced to proceed with great vigor against the admixture of all foreign materials to cement. This was done by making the membership of a cement manufacturer dependent on the following declaration, which he was required to sign:

"DECLARATION.

"(*a*) The undersigned members of the Association of German Portland Cement Manufacturers bind themselves to produce under the name of Portland cement only such an article as is made by calcining a thorough mixture, consisting essentially of calcareous and clayey substances, and then grinding the same to the fineness of flour.

"Any article made in a manner differing from the above method, or to which during or after burning any foreign substances have been added, is not recognized by them as Portland cement, and the sale of such products under the designation "Portland cement" is regarded by them as defrauding the purchaser. This declaration does not apply to such minor additions which are made to regulate the setting time of Portland cement, and which are permitted to an extent of 2 per cent.

"(*b*) A member acting contrary to the obligations assumed under *a* shall be disqualified from membership in the Association, and his disqualification shall be made publicly known.

"(c) In making this declaration the undersigned members recognize that the officials of the Association are in duty bound to see that the assumed obligations are adhered to."

In order to give this declaration real significance the association assumed the name "Association of German Portland Cement Manufacturers."

The officials of the Association have, since all members have signed it, made it a point to require that the obligations assumed by the signers are kept. This control, however, can only extend to the possible adulteration during or after burning, and to the raw material out of which Portland cement is to be made. To determine differences in quality of the various brands of Portland cement, a special test remains to be made now, as formerly.

The quality of Portland cement depends first on the combination of the raw materials, and after that on care in manufacture, the proper chemical combination, the degree of burning, the grinding, etc. The properties which are thus given to a particular brand of cement appear especially in the different setting qualities, the power of maintaining a fixed volume, the fineness of the powder and the tensile and compressive strengths; and the testing of these mechanical properties is imperative, in order to form a proper judgment of its qualities.

The remainder of this paper will be devoted to a comprehensive view of the testing methods now prescribed in the German Empire, as well as those in common use beyond the required ones. A description of the apparatus and machinery which has proved itself best adapted to this purpose, and is used chiefly at leading factories, will also be given. Want of space will forbid entering into a description of older or rarely used and unsuccessful apparatus. Tests, also, which require extended chemical knowledge and are incapable of being executed on the site of building by the engineer or architect, in case of need, cannot be considered. In connection with testing methods, it is intended to show, as far as possible from actual examples, the relation of German cement to them, and how the various methods were the actuating cause of a constant improvement of the product.

The regulations for uniform delivery and testing of Portland cement declare it to be a product created by burning to a clinker, a thorough mixture consisting essentially of clay and chalky substances, and then pulverizing the same to the fineness of flour. To regulate

important technical properties, foreign substances not exceeding 2% of the weight may be added without overstepping the regulations.

In the following paragraphs the most important of the regulations are given unchanged, and are in italic.

I. Packing and Weight.—As a rule, Portland cement must be packed in standard barrels containing 180 *kg.* (396 *lbs.*) *gross, and about* 170 *kg.* (370 *lbs.*) *net, and in half barrels of* 90 *kg.* (198 *lbs.*) *gross, and about* 83 *kg.* (183 *lbs.*) *net. The gross weight must be marked on the barrels.*

In case the cement is wanted in barrels of different weight or in sacks, the gross weight must likewise be plainly marked thereon.

Leakage, as well as possible variations in individual cases, are allowable to an extent of 2 *per cent.*

The barrels and sacks, besides the weight, must also be plainly marked with the name or trade-mark of the manufacturing firm.

In the interests of the purchaser and of sound business, the clause relating to an uniform weight is urgently needed.

For this purpose the most commonly used and almost exclusively recognized weight in international commerce of 180 kg. gross (about 400 lbs.) has been chosen.

II. Time of Setting.—According to the purpose for which it is intended, quick or slow setting Portland cement may be demanded. Slow-setting cements are those that set in about two hours or more.

In order to determine the time of setting of a cement, proceed as follows: Make a stiff paste of pure cement and water according to the regulation method, stirring the slow-setting cement three minutes and the quick-setting one minute. On a glass plate make a cake of this paste about 1.5 cm. (0.4 in.) thick, thinning towards the edges, putting all the cement necessary for this end on the plate at once. The consistency of the cement paste should be such that, after having been put on the plate by a spatula, it requires repeated jarring of the plate to cause the paste to thin towards the edges. To this end about 27 to 30% of water is sufficient. As soon as the cake has assumed sufficient hardness to bear the gentle pressure of a finger-nail, the cement is considered as beginning to set.

For an accurate determination of the time of setting and the beginning of the set, which is of importance with quick-setting cements (owing to the fact that cement must be used before it begins to set), it

is best to employ Vicat's regulation needle, endorsed by the Munich Conference and shown in Fig. 1. This needle, of circular cross-section, the end cut at right angles to the axis, having an area of 1 sq. mm. (.002 sq. in.), weighs 300 gr. (about 11 oz.). The tests are made as follows: A ring 4 cm. (1½ ins.) high and 8 cm. (3 ins.) diameter, made of a non-conducting and non-absorbent material, is placed on a glass plate, filled with cement paste of the above-mentioned consistency, and brought under the needle. The moment at which the needle no longer entirely penetrates the cement cake is taken as the "beginning of the set." The time elapsing before the needle ceases to make an impression on the hardened cake is taken as the "time of setting." To determine the proper consistency, let a rod of 1 cm. (0.4 in.) diameter replace the needle, giving a weight of 330 gr. (11.6 oz.). The consistency of the cement paste is to be such that the rod does not entirely pierce it.

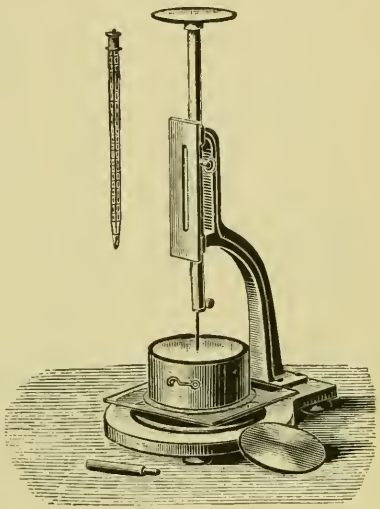


FIG. 1.

Since the setting of cement is influenced by the temperature of the air and water (high temperature hastening, low temperature retarding, it), it is desirable, in order to reach comparable results, to make the tests at a mean temperature of 15 to 18° Cent. Cements generally show a more or less marked rise in temperature while hardening, the maximum being coincident with the beginning of hardening. During the setting, slow-setting cement should not become markedly heated, but quick-setting cement may show a considerable rise of temperature.

Through long storage Portland cement, as a rule, becomes slower setting. It gains in activity if kept in a dry place free from drafts, becoming at first quick setting, and after a few months gradually slow setting. How this varies among different cements is shown in the following table, which is an extract from a complete statement on this point, laid before the General Assembly of the Association

of German Portland Cement Manufacturers on March 24th, 1893, by Dr. Tomëi.

Brand of Cement.	Beginning of time of setting in hours and minutes.								
	Fresh.	Months.							
		1	2	3	4	5	6	9	12
Cement I, in barrel	4.00	3.00	.....	0.10	0.30	0.25	1.00	4.30	5.00
Cement I, in sack	5.00	0.15	0.25	1.25	1.00	2.30	2.13	4.00	5.00
Cement I, with ¼% of plaster of paris	5.00	6.00	.....	6.00	1.00	0.30	1.22	4.00	5.00
Cement I, with ½% of plaster of paris	4.00	4.00	5.00	3.40	3.30	.....	4.30	3.45	5 00
Cement II	3.30	.....	0.20	.....	.....	6.00	.....	6.00	4.00
Cement II	2.30	2.15	0.15	0.23	.....	2.00			
Cement III	5.00	.....	.....	0.11	....	4.00	.....	4.00	4.00
Cement IV	4.30	5.00	5.30	0.20	0.14	0.18			
Cement V	4.30	4.00	3.00	5.00	5.45	0.40			

The still-prevalent idea that Portland cement deteriorates from long storage is an erroneous one, and all contracts specifying the use of a fresh product only should be discountenanced. The setting qualities of a cement are not only to be considered in its choice for a certain purpose, but also in judging it from the ultimate strength as determined through the regulation testing methods. Of two quick-setting cements, the quicker will be the stronger in 24 hours after hardening, but in 28 days the slower will have the lead. From this point the first begins again to approach the latter and sometimes even passes it. For a mortar of which immediate strength is demanded, or for béton to be used under water, the quick-setting cement should, therefore, always be used; otherwise the slow-setting variety should have the preference, owing to its generally greater strength as well as certainty of maintaining a constant volume.

The quantity of water used in mixing cement has a decided influence in the beginning of the set. To regulate this it is necessary always to bring the mortar to the same consistency. Based on many years of experience, Böhme recommends $(Q\text{–}4)\%$ of water, in which Q denotes the percentage of water necessary to bring the pure cement to the consistency of syrup. The cement paste has the regulation viscosity, measurable by Vicat's apparatus.

III. Constancy of Volume.—The volume of Portland cement should remain constant. The decisive test of this should be that a cake of neat

cement, made on a glass plate, protected from sudden drying and placed under water after 24 *hours, should show, even after long submersion, no signs of crumbling, or of cracking at the edges.*

In carrying out the regulation test for constant volume the cake made for the purpose of ascertaining the time of setting is placed under water. With a slow-setting cement this is done after 24 hours, and in no case before the beginning of setting. For quick-setting cements only a short time is necessary. It is essential to protect the cakes, especially those of slow-setting cements, from drafts and sunshine until the setting has begun. This is effected by keeping them in a covered box or under moist cloths. The formation of drying cracks is thus avoided.

If crumbling or cracking at the edges appears while the cement is hardening under water, swelling is undoubtedly indicated, *i. e.*, in consequence of an increase in volume a gradual loosening of the bond first formed takes place, and this may lead to a total disintegration of the cement. Indications of swelling generally appear at the end of three days; at any rate, observations extending over 28 days satisfy the regulations. Care must be taken, in making tests for constancy of volume, to distinguish between cracks due to swelling or contracting, and such as are due to a too early submersion of the samples. Since Portland cement naturally increases slightly in volume while hardening under water (which increase rapidly disappears on drying), so-called hair or air cracks may appear as a consequence of repeated change of the cement from a dry to a wet state. These cracks are, however, of detriment to building only when the cement is not properly used. Considerable changes in volume, by swelling of the cement, should, however, be attributed to inferior quality. These changes consist in a considerable expansion which effects a dissolution of the bond previously formed. The swelling takes place after the beginning of setting; the stronger the activity, the earlier the swelling commences. On cement cakes immersed in water it may plainly be seen at the end of a few days in the appearance of a network of fine cracks or, if very marked, in crumbling and cracking of the edges. The cracks around the edges radiate in wedge-shape from the center of the cake. On the contrary, cracks due to shrinkage form irregular curves crossing themselves. Very slow-setting cements are predisposed to such cracking, these forming even at the beginning of setting, especially if

the cement is then exposed to a draft or to sunshine. Cracks caused by putting the cement under water before the beginning of setting should not be confounded with those due to swelling.

One drawback to the method described above lies in the length of time requisite, to shorten which artificial heat has been used to harden the cakes in air instead of allowing them to harden under water. Under this head may be classed the earlier methods from England; the use of balls, according to Heintzel; the test of boiling, according to Michaelis; the steam test of Heintzel, and the lately again-recommended high-pressure test of Erdmenger. The last-mentioned test is the subject of a special paper: "Report on the Investigations Made to Determine the Effect of Magnesium on Portland Cement," by Dr. L. Erdmenger, Hanover, 1893. In North America the hot tests have lately received due attention and been recommended by Mr. W. W. Maclay, M. Am. Soc. C. E.* According to experience in Germany, however, all tests save the German method of cakes are apt to mislead, and we have therefore been obliged always to return to the standard test for constant volume as the most reliable one. In fact this test has caused the occurrence of swelling cements to be of the very greatest rarity in late years. The tests made at the Royal Prussian Station for Testing Building Materials gave the following results:

In the year—	Number of cements tested.	Number not of constant volume.	In the year—	Number of cements tested.	Number not of constant volume.
1879-80	25	0	1885-86	113	1
1880-81	43	0	1886-87	72	
1881-82	83	0	1887-88	103	
1882-83	63	0	1888-89	136	4
1883-84	80	0	1889-90	113	2
1884-85	99	0	1890-91	119	0

Cements of changeable volume differ also in other properties, especially in tensile strength, from the Portland cements, so that they are easily recognized. Although the statement has been made above that the ordinary "swelling cements" may be readily recognized after a few days or at the most weeks of hardening, through the standard tests for constant volume, the cements with a high percentage of magnesia, when burned like Portland cement to a

* "Hot Tests for Determining Change of Volume in Portland Cement." *Transactions* Am. Soc. C. E., Vol. XXVII, p. 412.

slag, seem to form an exception. In such cements swelling appears under water at the end of about a year; in air frequently later, but with increased violence. Mr. R. Dyckerhoff and others have made extensive experiments on this point, employing for this purpose Bauschinger's caliper apparatus. This apparatus, shown in Fig. 2, is employed to advantage wherever it is desirable to follow accurately the

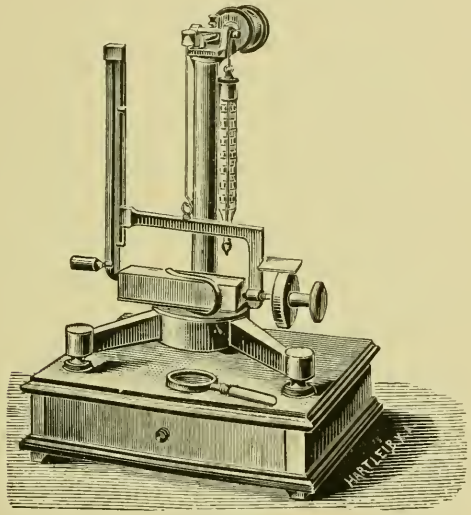


Fig. 2.

expansion or shrinkage of a specimen over an extended period of time. It enables us to determine by direct measurement the changes in length of small parallelopipedons of about 100 mm. (4 ins.) long and 5 sq. cm. (0.78 sq. ins.) area with an accuracy of $\frac{1}{200}$ mm. ($\frac{1}{5000}$ in.). The apparatus consists principally of a stirrup-shaped caliper, having a fine micrometer screw on its right arm, the left being the support of a sensitive lever. The shorter arm of the lever terminates in a blunt caliper point, and is pressed against the measuring screw by a spring attached to the long arm. The calipers are readily moved in any direction, and the micrometer is read in the usual manner. One revolution of the screw equals 0.5 mm. ($\frac{1}{50}$ in.), and readings on the head are made at $\frac{1}{200}$ mm. ($\frac{1}{5000}$ in.). The specimen is placed on a small platform, between the lever and the screw. The points of the calipers are set on center marks drilled into small glass plates let into the specimens.

The width between the caliper points is made equal to 95 mm. (3¾ ins.) in each of these instruments, thus very much simplifying the computations for length. For instance, if the screw reads 9.56 revolutions, the absolute length of the specimen is $\frac{9.56}{2} + 95.00$ mm. $= 99.78$ mm.

The specimens are made in small metal frames, just as the standard specimens for tension. It is necessary, however, to turn the moulds over repeatedly, and treat both the upper and under surfaces alike. If this is not done, and the upper surface becomes rather thick and smooth, which a repeated striking off with the trowel will accomplish, it may easily happen that the lower layers remain loose and porous, causing a distortion of the specimen, which may lead to considerable errors. The positions for the center-mark plates are provided for in the forms, and these plates may, therefore, be cemented into place as soon as the specimens are removed from the moulds. To measure a specimen requires but a few minutes, the apparatus being very easy to manipulate.

Long series of experiments have been made in Germany with this apparatus, the results being published in the "Proceedings of the Association of German Portland Cement Manufacturers." An unpublished series of such experiments has been made by the manager of the Portland cement factory "Stern," Dr. Tomëi.

In the following table, F and G are two cements which were tested for tensile strength in a 1:3 mortar, and showed but small strength. It will also be noted that these two inferior brands showed an extraordinary degree of shrinkage, making them unfit for decorative purposes and laying of face stones. This extraordinary shrinkage explains the cracks shown on so many ornamental surfaces, artificial stones and plates, which always have either a neat cement or a mixture low in sand at their surface. The preference for a really good brand of cement for this purpose is thus explained. The table furthermore shows that the commonly accepted theory regarding a uniform relation between expansion when hardening under water and shrinkage when hardening in air is erroneous. The various cements found in the market vary widely in this respect. This much, only, may be said—in general, cements having low compressive strength are liable to great changes in volume when setting in air. Cements not manufactured according to the standards also show great changes in volume.

Portland Cement. Brand.	Hardened in Air.						Hardened in Water.					
	Neat cement.			Normal mixture, 1 to 3.			Neat cement.			Normal mixture, 1 to 3.		
	Time of Hardening, in Days.											
	7	28	90	7	28	90	7	28	90	7	28	90
A. Ground to ordinary fineness	— 0.055	— 0.121	— 0.134	— 0.019	— 0.034	— 0 086	+ 0.021	+ 0.037	+ 0.048	+ 0.006	+ 0.012	+ 0.033
A. Not so finely ground	— 0.056	— 0.106	— 0.160	— 0.010	— 0.043	— 0.060	+ 0.027	+ 0.027	+ 0.003	+ 0.012	+ 0 017	+ 0.020
B	— 0.101	— 0.188	— 0.260	— 0.025	— 0.074	— 0.123	+ 0.003	+ 0.013	+ 0.010	+ 0 005	+ 0.033	+ 0 035
C	— 0.075	— 0.118	— 0.175	— 0.015	— 0.020	— 0.068	+ 0.005	+ 0 020	+ 0.010	+ 0 030	+ 0.005	+ 0.030
D	— 0.055	— 0.125	— 0.185	— 0.015	— 0.073	— 0.090	+ 0.005	+ 0.012	+ 0.019	+ 0.005	+ 0.010	+ 0.005
E	— 0.045	— 0.120	— 0.175	— 0.023	— 0.075	— 0.110	+ 0.027	+ 0.048	+ 0.035	+ 0.007	+ 0.035	+ 0 043
F	— 0.117	— 0.252	— 0.321	— 0.032	— 0.072	— 0.120	+ 0.035	+ 0.035	+ 0.053	+ 0.015	+ 0.020	+ 0.019
G	— 0.125	— 0.234	.....	— 0.045	— 0.058	.....	+ 0.025	+ 0.035	.....	+ 0.013	+ 0.020	.....

For the sake of illustration a number of such cements, used by R. Dyckerhoff, Amöneburg, to test the effect of magnesium, have been noted (Fig. 3). In passing it may be mentioned that Dyckerhoff, in agreement with Prof. Débray, of the École des Ponts et Chaussées, Paris, Dr. Erdmenger and others, found a heightened tendency to

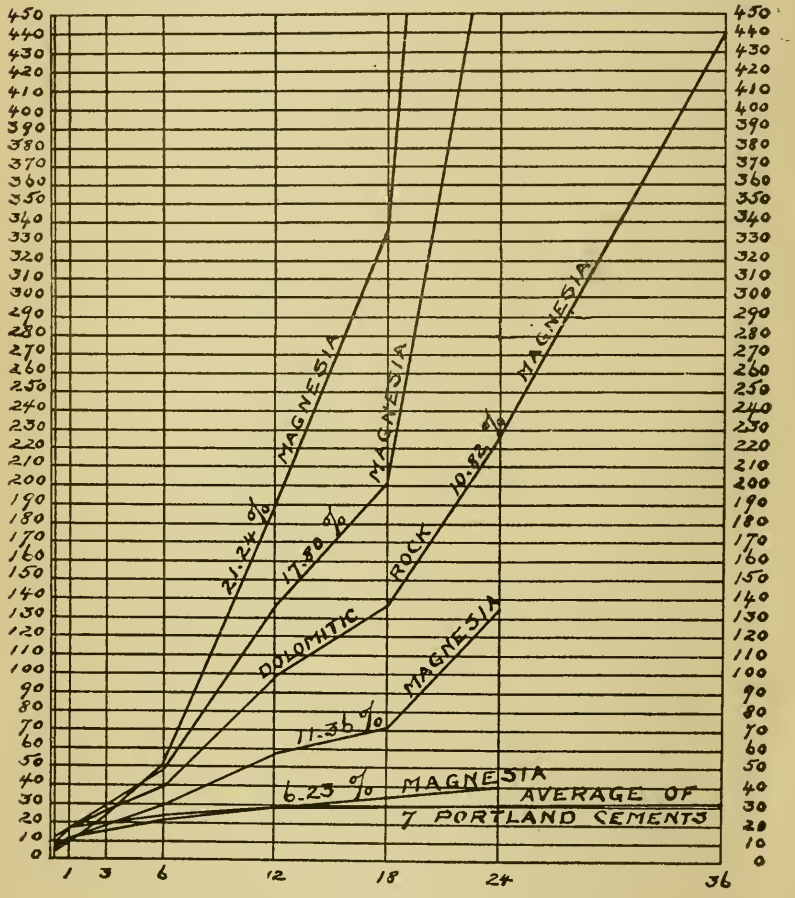


FIG. 3.

swelling (variable volume) in cements containing much magnesia, and also that the maximum amount of magnesia a cement may safely contain is not yet determined. In Germany a special commission is still engaged in further detailed investigations. It is, however, certain

that a cement may contain as high as 5% of magnesia without causing apprehension as to its qualities.

IV. Fineness of Grinding.—Portland cement must be ground so fine that no more than 10% *of a sample shall pass through a sieve of* 900 *meshes per square centimeter* (5 800 *per square inch*). *The thickness of the wires of the sieve to be one-half the width of the meshes.*

The quantity of cement to be used for this test is 100 gr. (3.2 oz.). Since cement is frequently used with a high percentage of sand, and the strength of mortar increases with the fineness of the cement used (because then more parts of the cement come into action), the fineness of cement is of a high value. It therefore seems desirable to test the fineness of the grain in a uniform manner by means of a sieve of the gauge given above. In Germany, brass-wire sieves having 180, 324, 600, 900, 4 900 and 5 000 meshes, respectively, per square centimeter are used (1 200, 2 200, 3 800, 5 800, 31 000 and 32 000 meshes per square inch). The sieve of 900 meshes is, according to the regulations, the standard.

It would be fallacious to decide from the fineness alone of a cement as to its quality, since inferior, soft cements are much oftener found finely ground than well burned. Well-burned cements will show a higher strength, as a rule, even if not so well ground as those poorly burned. If the cement is to be used mixed with lime, it is best to employ a well-burned and finely ground quality, the increased cost of which is counterbalanced by a marked improvement of the mortar. Since a well-burned, finely ground cement possesses a greater value, it is easily seen why the manufacturers have aimed to attain a high degree of fineness, while maintaining a high specific gravity. The following table will show to what extent the perfecting of old and the introduction of new mill machinery have produced the desired end. From it also appears how the cements tested at the Royal Prussian Testing Station for Building Materials approached the standards more nearly as the years went by. Only isolated cases of cements leaving more that 20% on the sieve of 900 meshes occur in the 12 years given in the table. Cements leaving between 20 and 10% have increased in number since 1882, and in the last few years very decidedly, while the very fine ones leaving less than 10% have increased proportionately. This increase is especially marked since 1887, the year of introducing the new standards, and shows that suc-

cess has not failed to crown the efforts of the German cement manufacturers to perfect their product. The year 1890–91 did not show a single one of the 119 samples of cement tested, both officially and privately, to fall short of the requirements of the regulations regarding fineness.

FINENESS OF GERMAN PORTLAND CEMENT.

CEMENTS TESTED.		Above 20 per cent.		Between 20 and 10 per cent.		Below 10 per cent.	
		Failing to pass through 900-mesh sieve.					
Years.	Number.	Number of cements.	Percent.	Number of cements.	Percent.	Number of cements.	Percent.
1879–80	25	2	8.0	13	52.0	10	40.0
1880–81	43	.....	.....	6	14.0	37	86.0
1881–82	83	1	1.2	29	34.9	53	63.9
1882–83	63	1	1.6	25	39.7	37	58.7
1883–84	80	4	5.0	26	32.5	50	62.5
1884–85	98	3	3.1	39	39.8	56	57.1
1885–86	115	.....	.....	37	32.2	78	67.8
1886–87	72	1	1.4	16	22.2	55	76.4
1887–88	105	3	2.9	13	12.4	89	84.8
1888–89	146	.....	.....	3	2.0	143	98.0
1889–90	113	.....	.....	3	2.7	110	97.3
1890–91	119	.....	.....	.....	.....	119	100.0

V. Tests for Strength.—The binding strength of Portland cement is to be determined by testing a mixture of cement and sand. The test is to be conducted for tensile and compressive strength according to a uniform method, and is to be performed upon test specimens of like form, like cross-section, and with like apparatus. It is recommended, besides, to determine the strength of neat cement.

The tests for tension are to be made upon briquettes of 5 *sq. cm.* (0.78 *sq. in.*) *cross-section at the place of rupture, the tests for compression upon cubes of* 50 *sq. cm.* (7.8 *sq. ins.*) *area.*

Since experience has shown that the results for tensile strength obtained from neat cements do not give uniform conclusions regarding the binding properties of the cement and sand, especially where brands from different factories are under comparison, it is recommended to test the binding strength of Portland cement by means of a sand mixture. The testing of cement without sand is to be especially recommended if a comparison of Portland cement is to be made with mixed

cements and other hydraulic binding mixtures. In this way the superiority of Portland cement, owing to certain qualities which are wanting in the other hydraulic binding materials, is better brought out than when tested as a sand mixture.

Although the relation of compressive to tensile strength is a varying one among hydraulic cements, yet the test for tensile strength is frequently the only one used in comparison of different kinds. This, however, leads to an erroneous judgment of them.

And further, since in practice mortar is chiefly called upon to withstand pressure, the standard test of strength should be no other than the test for compression. In order to maintain uniformity in these tests the German regulations recommend the use of such apparatus and tools as are employed at the Royal Testing Station at Charlottenburg, Berlin. These instruments will be further described below.

VI. Tensile and Compressive Strength.—Slow-setting Portland cement, when mixed with standard sand in the proportion of 1 *part of cement to* 3 *of sand by weight,* 28 *days after being mixed—one day in air and* 27 *in water—must possess a tensile strength of not less than* 16 *kg. per square centimeter* (225 *lbs. per square inch*), *and a maximum compressive strength of* 160 *kg. per square centimeter* (2 250 *lbs. per square inch*).

Quick-setting cements generally show a lower strength after 28 *days than that given above. The time of setting must, therefore, be given when stating figures relative to strength.*

When comparing several varieties of cement it is absolutely necessary to make a test with a high percentage of sand, because the setting properties of different varieties vary much with the amount of sand. A proportion of 1 part cement to 3 of sand by weight is taken as a standard, for with 3 parts of sand the binding qualities of a cement are sufficiently tested.

A cement which shows a higher tensile or compressive strength often allows a larger percentage of sand, and, on this account, as well as owing to greater strength with the same amount of sand, can fairly command a higher price.

The standard-strength test is that for compression after an interval of 28 days. In a shorter period the time of setting is not sufficiently emphasized. The results may, for instance, show an equal strength for different cements after 28 days, while at the end of but seven days there would be a marked difference.

The tensile strength, after 28 days, serves as the usual test of the furnished product. Should it, however, be desirable to make a test at the end of seven days, the relation of the tensile strength after seven days to that at the end of 28 days must first be determined. This test may also be made with neat cement, if the relation of the strength of cement to that with 3 parts sand and 28 days old has been determined. It is recommended that, whenever possible, the tests be extended over some time, the briquettes being made and kept for this purpose, in order to determine the behavior of different cements after a longer period of hardening.

The standard sand used at the Royal Testing Station in Berlin is produced by washing and drying quartz sand which must be as clean as possible, and afterwards be sifted through a sieve of 60 meshes per square centimeter (387 meshes per square inch), by which process the coarser particles are separated. The sand is again sifted through a sieve having 120 meshes to the square centimeter (774 per square inch). The residue remaining in this sieve is the "standard sand" for experiments, the coarsest and finest particles having been eliminated. It is absolutely necessary, in order to obtain uniform results, to use only the standard sand described above, as the size of grain of the sand has a material influence on the results of the testing. The standard sand must be clean and dry, and all earthy and other substances be previously removed by washing.

The Prussian sand is obtained near Freienwalde on the Oder, in the province of Brandenburg, and may be had sifted and washed from the Chemical Laboratory for Clay Industry, No. 6 Kruppstrasse, Berlin.

Since all quartz sand does not give the same strength even if uniformly handled, it becomes necessary to ascertain if the sand at disposal is up to the standard furnished under direction of the management of the "Association of German Cement Manufacturers," which is the standard used at the Royal Testing Station at Berlin. Sand possessing the standard qualifications is found in various parts of Germany. An international standard sand has not yet been adopted, and it is on this account that tests made on Portland cement give different results in different countries. The Russian standard sand, for instance, gives markedly lower, and the Swiss sand considerably higher, strength than the German.

In order to avoid the variation produced in the sand by the gradual

change in sieves of wire netting, due to wear, the last conference for the unification of testing methods, held in Berlin, 1890, decided to substitute for the wire sieves others of tin, the openings being circular. A commission was appointed to fix the size of the holes and their arrangement, as well as the thickness of the tin. The new sieves must separate a sand which shall possess a tensile strength equal to that furnished by the present standard sand. It is expected that a decision will be reached on this point at the conference to be held in Vienna in 1893.

Tensile Tests.

(*a*) *Handwork.*—On a slab of metal or marble are laid 10 small sheets of filtering paper, which have been previously saturated with water, and upon these are placed 10 brass moulds, thoroughly clean and moistened with water, 250 gr. of cement and 750 gr. of dry standard sand are thoroughly mixed, and to this is added the requisite quantity of water as specified by weight, in the most cases 10% = 100 gr. The whole mass is then worked up with a trowel or spatula until it exhibits an appearance of perfect homogeneity. In this manner a very stiff mortar is obtained, which has the appearance of freshly dug moist earth. With this mortar the 10 moulds are filled and heaped up; the mortar is then beaten into the moulds with an iron trowel (of from 250 gr. (= 10) to 13 ozs. weight), at first lightly and afterwards more heavily, until it becomes elastic and water appears on the surface (such a trowel is pictured in Fig. 5). It is of the utmost importance that the blows be kept up uninterruptedly until this point be attained. The superfluous mortar is now scraped off with a knife, and by means of the same the surface is leveled. By using greater force a larger quantity of mortar than specified above might be hammered into the moulds, and the briquettes gain in tensile strength accordingly; this would, however, be against the regulations for the uniform delivery and testing of Portland cement.

When the briquettes have become sufficiently hard, they are taken out of the moulds by undoing the screws, and any adhering filtering paper is removed. After remaining 24 hours exposed to the air, at a temperature of about 60° Fahr. and in a covered box, the briquettes are immersed in water having the same temperature, and care must be taken that they remain covered with water until the time arrives for taking them out. On the day of testing the briquettes are taken from

the water and immediately broken on the machine. The average of 10 breaking weights furnishes the strength of the mortar tested.

(*b*) *Machine Work.*—The specimens are to be made with the hammer apparatus of Dr. Böhme, and the hammer of 2 kg. ($4\frac{1}{2}$ lbs.) weight is

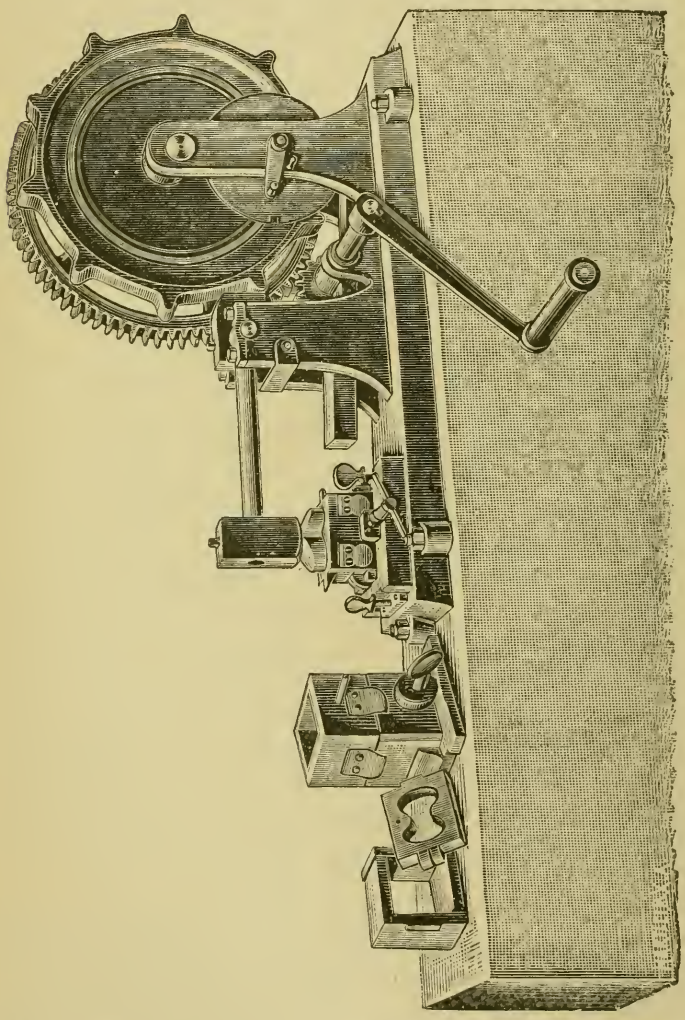


Fig. 4.

to deliver 150 blows upon the mould. If the prescribed directions are closely followed the results obtained from machine and hand made specimens agree well.

For a disputed case, however, the machine-made specimens are to be considered the standard. Böhme's hammer apparatus (Fig. 4) is a tilt hammer with automatic action.

The hammer is driven by a cam wheel of 10 cams actuated by a simple gearing. The wrought-iron handle of the hammer is let into the cross-head which carries the axle of the hammer and keyed to this cross-head and to the cap, so that it may be readily replaced if worn. The steel hammer of 2 kg. (4½ lbs.) is similarly fastened to the cap. As soon as the intended number of blows has been delivered, the mechanism is automatically checked, the proper setting having been made for this purpose before beginning the work.

The forms to receive the mortar (which are shown separated and together for tensile and compression specimens in Fig. 4) consist of a lower and upper case held together by means of springs. The lower case for compression specimens consists of two angle irons held on a planed plate by a grinding strip and a screw acting on the latter. Upward motion is prevented by two wedge-shaped surfaces. The lower case and half of the upper one is filled with the mortar to be tested, and a plate laid upon its surface. On this plate the blows are delivered. It is of vital importance that the apparatus rest on a firm non-elastic foundation, preferably it should be placed and fastened on a pier of masonry.

Compression Tests.—In order to have tests for compression made at different places give results in agreement with each other, it is necessary to make the specimens by machinery.

Take 400 gr. (14 ozs.) of cement and 1 200 gr. (42.2 ozs.) of dry standard sand, mix thoroughly in a dish, add 160 gr. (3.6 ozs.) of water, and work the resulting mortar thoroughly for five minutes (quick-setting cements are to be worked but one minute). Put 860 gr. (30.3 ozs.) of this mortar into the cubic mould properly provided with filling cases and fastened to the bed plate. The iron core is placed into the form, and 150 blows are delivered on it by means of the hammer apparatus, with the hammer of 2 kg. (4½ lbs.) weight. The filling cases and core having been removed, the specimen is struck off and flush, smoothed and drawn off the bed-plate together with the mould.

For neat cement specimens mix about 1 000 gr. (2.2 lbs.) cement with the requisite amount of water, and proceed as in *a*. The moulds

should be oiled a little, and can be removed only after the cement has sufficiently hardened.

There are no fixed directions in the regulations regarding the time the test specimens are to remain in the moulds, but it has been agreed that specimens for compression, as well as hand-made tensile specimens, are to be removed only after the mortar has well set, *i. e.*, from 20 to 24 hours after being made. Tensile specimens, machine made, may be at once taken from the moulds, since they are somewhat harder. The amount of water used is always to be mentioned when giving figures regarding strength.

Method of Making the Test.—All specimens are tested as soon as taken out of water. In tensile tests the increase in tension is to be made at the rate of 100 gr. (3.5 ozs.) per second, since the time occupied in making the tests is of importance.

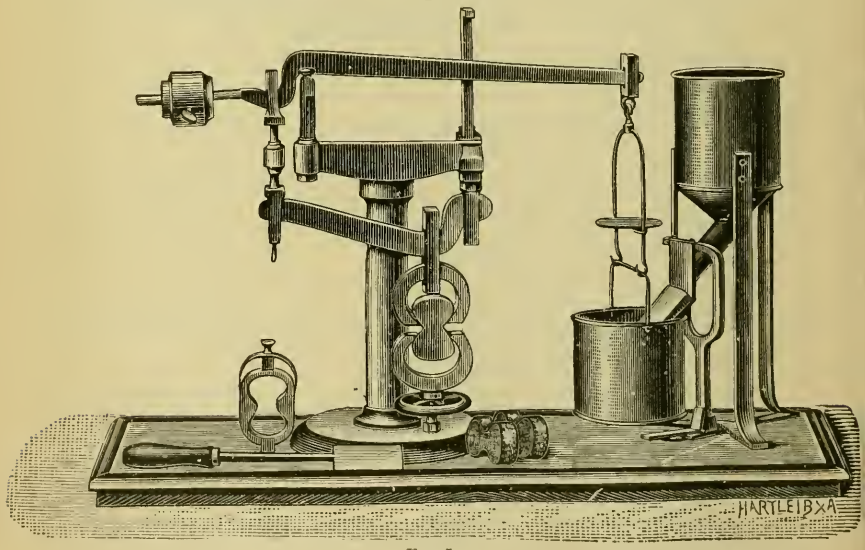


Fig. 5.

In order to insure uniform results, the pressure in compression tests is to be applied to the sides of the cube, and not to the bottom or to the top surface which has been worked. The mean of every 10 tests is taken, according to the regulations, as a measure of the strength, which the resolutions of the Munich Conference prescribe every six specimens, of which the arithmetic mean of the highest four is taken as the measure of strength. Dr. W. Michaelis' double-lever automatic

cement-testing machine is the one universally employed in Germany for making tensile tests. This machine is shown in Fig. 5.

Upon a massive pillar, about $\frac{1}{3}$ m. in height, are fastened two levers connected with one another; the upper has a leverage of 10 to 1, the lower of 5 to 1. To the latter is fastened the upper clip; the lower clip is attached by means of a ball joint to a screw, with a hand-wheel for raising or lowering. The clips are constructed on the most scientific principles and insure the strain being brought to bear on the briquette in the most favorable manner. There is also a counter balance for bringing the levers into exact equilibrium. At the time of testing the briquettes are taken directly from the water and set up on edges to drip. The lower clip is then raised by means of the wheel, and the briquette is placed in position. The adjustment of the levers is now effected by means of the wheel. In adjusting the briquette, care should be taken that the upper clip is exactly over the under one, and that their ends are horizontal and parallel; and that, after attaching the bucket for receiving the breaking weight, this latter should remain suspended 5 to 10 cm. above the level of the table. The vessel for the delivery of the shot is provided with a trap, which cuts off the stream of shot instantaneously. The shot is allowed to pour out by opening the trap.

At the moment the fracture occurs, the stream of shot instantly ceases. The bucket is now removed and weighed on a spring scale. The lever proportion is that of 50 to 1, and the section of the briquette 5 sq. cm.; the weight employed is, therefore, $\frac{5}{5\,0}$ of the actual breaking strain; the number indicated in the scale has thus to be multiplied by 10. Although the apparatus itself can be made to serve for ascertaining the weight employed in fracturing the briquette, the use of a spring scale is recommended, on account of the saving in time it effects, and because, thereby, a direct reading is possible. If it be desired to ascertain the breaking weight by means of the apparatus itself, the bucket, with the shot, is suspended from the hook, which will be found under the connecting piece, between the two levers; the upper one of these is brought into equilibrium by placing weights in the pan at the farther end. The breaking weight employed is thus again weighed decimally, the number of grams required to counterbalance it representing $\frac{1}{1\,0\,0}$ of the actual breaking strain per sectional unit; the actual breaking weight

in kilograms per square centimeter is obtained by simply cutting off the last place from the number of grams (placing the decimal point before the last figure). In order to test accurately very weak briquettes it is sufficient to place 10 to 20 grains of shot in the bucket, that the clips may grip the briquette. The remainder of the weight is then added in the form either of fine shot or pounded quartz, dropped in from the hand or from a vessel with a spout supplied for the purpose. The weight is best ascertained by means of a small hand balance; the weight indicated must be multiplied by 10, as above stated.

Testing by Compression.—Portland cement ceases to grow in tensile strength after some years; its elasticity decreases while its compressive resistance increases; it becomes like a piece of pottery, and should then be tested by compression. If it be remembered that in order to crush a cube of only 50 sq. cm. a great weight is necessary, it will be obvious that only powerful presses can be used.

Figs. 6 and 7 illustrate a machine for testing the compressive resistance of cement. This machine is a hydraulic press, the pressure of whose liquid is so far reduced by a system of pistons that it can be easily measured by the pressure of a column of quicksilver of convenient height. In Fig 6 A is the piston causing the pressure, B and C are the pistons to reduce the pressure of A, D is the quicksilver manometer, the chief part of which is a glass tube open at the top, standing at the bottom in connection with the space under the piston C. The cube E lies between the two plates F and G, of which the first rests with a spherical surface on the piston A and can place itself in the proper position. The upper plate G hangs to the lower end of the screw H, and can be raised to a convenient height by means of a hand wheel. The cylinder K, in which the piston A moves, is filled with castor oil. If the piston-rod L is pressed into the cylinder K, a pressure is exerted on the fluid, and thereby the piston A lifted and the thin piston B driven downward. The piston B presses on the larger piston C, and this again exerts a pressure on the liquid underneath. This liquid consists of a layer of castor oil under the piston C, having only the purpose to render the piston air tight. Under the castor oil is quicksilver that fills the lower part of the cylinder M and the connection of the quicksilver manometer. In consequence of the pressure on the liquid under the piston C, the quicksilver rises in

the glass tube till the column balances the pressure. On the scale-board to the right of the glass tube the total pressure of the piston *A*

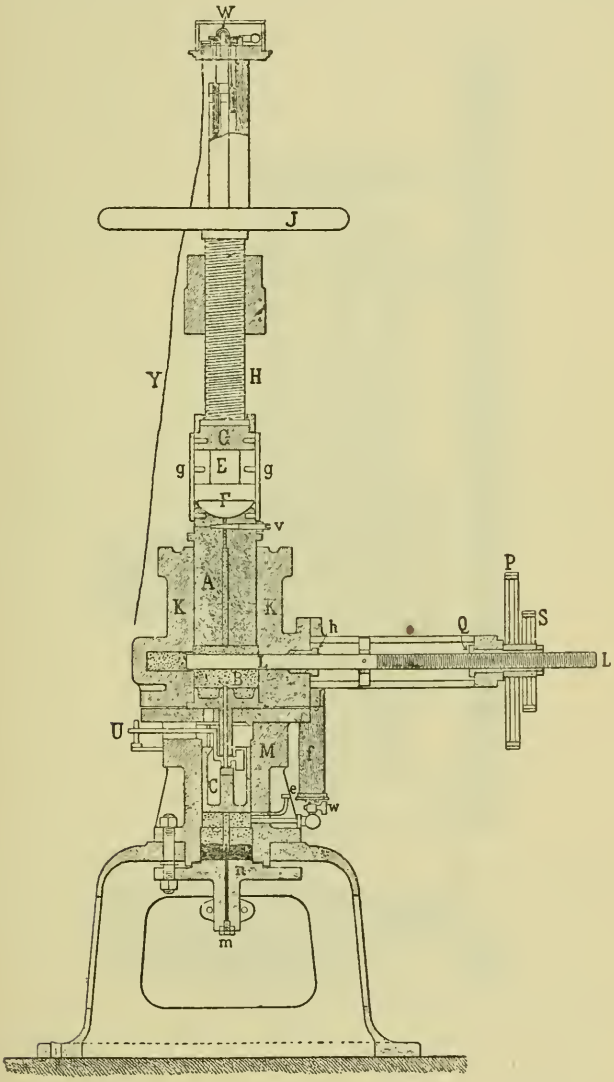


Fig. 6.

is read in tons (1 ton = 1 000 kg.); on the scale-board to the left of the glass tube the pressure exercised on 1 sq. cm. of the surface of a

normal cube (50 sq. cm.) is read in kilograms (instead of this the pressure of the liquid under A could be read in kilograms per square centimeter or in atmospheres). The maximum pressure of 30 000

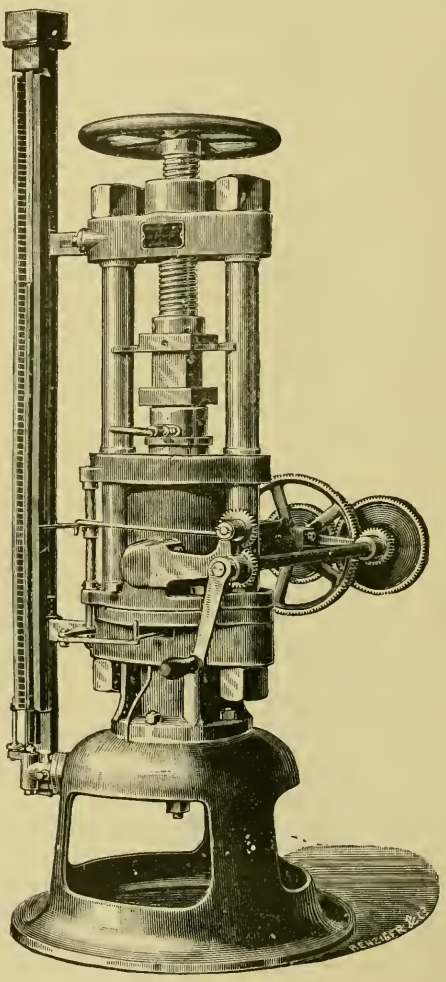


FIG. 7.

kg. on the piston A is equal to a height of the quicksilver column of about 140 cm.

A cheaper grade of the same apparatus is made in a slightly different form, but such accurate results cannot be obtained with it.

The upper plate of this apparatus shown in Figs. 8, 9 and 10 is rigid, and only the under one is attached by a ball and socket joint to the piston. The provision for measuring the intensity of the pressure is as follows: The large pressure cylinder *A* has hydraulic connection with a manometer *C* by means of a pipe *B*. This manometer consists simply of a thin piston, ground to fit, its upper end pressing against a plate and thereby tending to elongate a spring. The elongation of the spring is, therefore, proportional to the pressure existing in the large cylinder and consequently proportional to the pressure exerted on the cube of cement. This pressure may be read on a disc with two

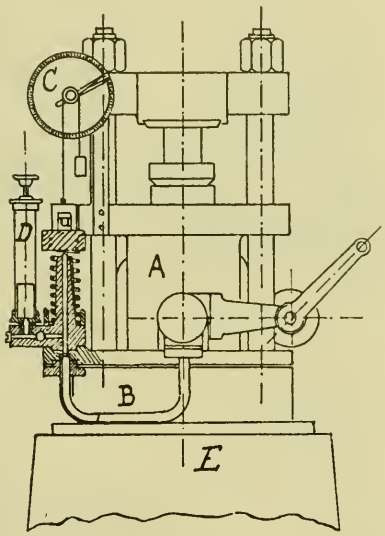


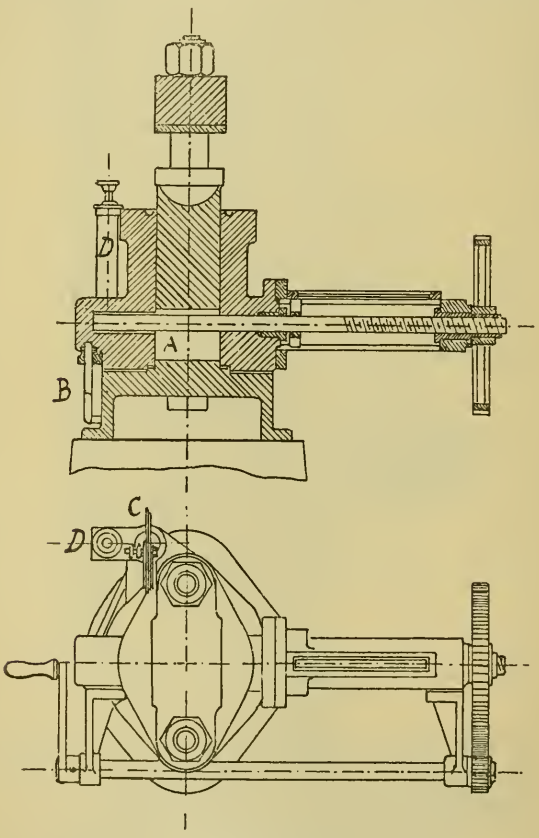
FIG. 8.

indices. The one is moved by a cord from the spring, while the other is carried along by the first, and remains wherever it happens to be when the specimen fails.

The press is driven by means of a crank, just as in the one previously described; but, to simplify, the construction, the device for the quick return of the pressure rod has been omitted. A hand pump *D* and a cock serve to raise and lower the under plate rapidly. A controlling lever, which may be removed when the apparatus is in use, serves to place the index in the proper position by means of directly attached weights.

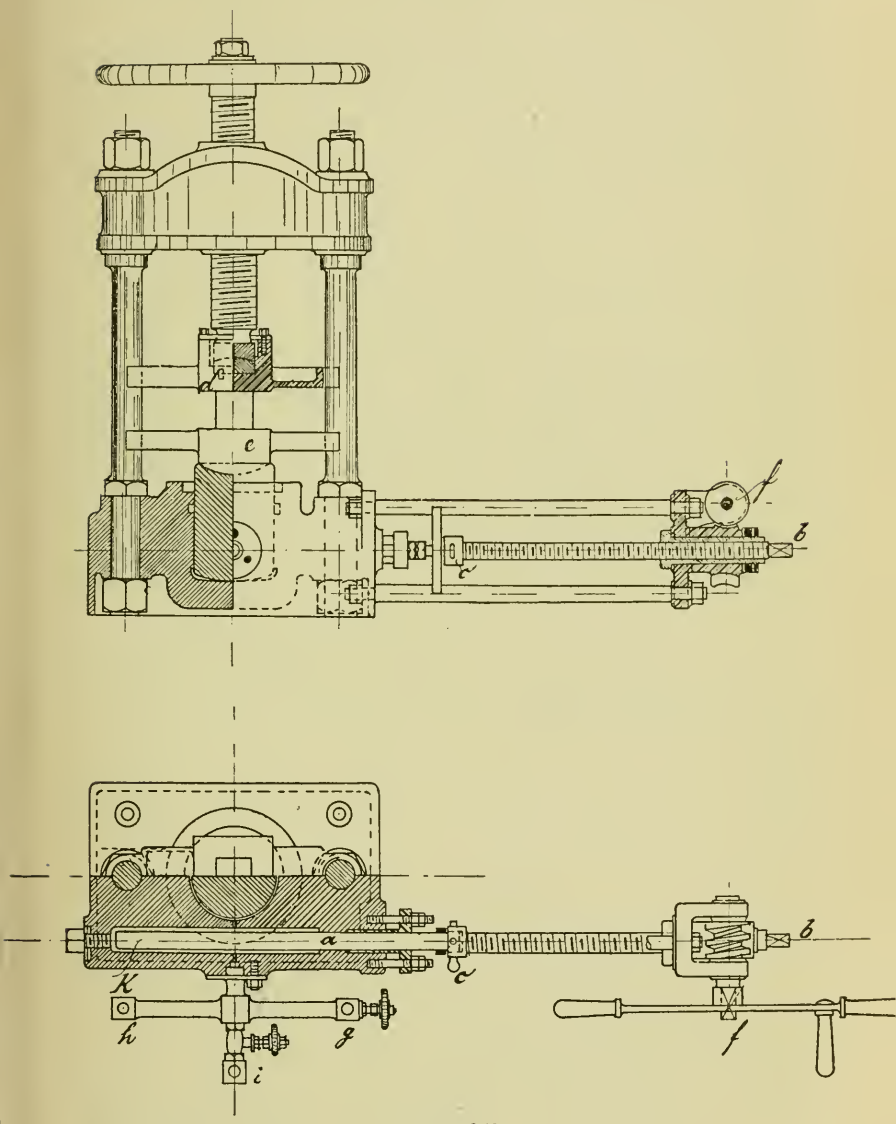
To test higher strengths, recourse must be had to a hydraulic press with metal manometer, such as are used for the most varied industrial purposes. Its construction is shown in Figs. 11 and 12.

The method of working this machine is as follows: The crank is put on at *b*, the coupling pin *c* removed and the piston *a* unscrewed completely. The body to be tested is then placed between the ball-



FIGS. 9 and 10.

bearing plates *e* and *d*, the coupling replaced, and the piston *a* turned by means of the crank placed at *f*, until the index of the manometer *g* or *h*, by remaining stationary or moving backwards, shows that failure of the test specimen has begun. The low pressure manometer *g* is to be kept closed if it is intended to use a higher pressure. The check manometer *i* is opened only by means of a check cock at *h*. The



FIGS. 11 and 12.

press is filled with glycerine which may be introduced above the piston *a* at about *K*.

The press described as the standard Swiss apparatus leaves nothing to be desired in accuracy of measurement. In this ring packing is not needed, and the piston friction is annulled by the use of castor oil. Lately another machine has been built by Engineer Suchier, of Frankfort-on-the-Main, which allows tests of tension, compression and adhesion. This is operated by oil under hydraulic pressure.

The perfection of cement-testing machines was the natural consequence of the altered development which the manufacture of cement in Germany took after the promulgation of the new standards, and which laid greater and greater importance on the highest possible compressive strength. The following tables give the experiments of the Royal Testing Station at Berlin. Considering the recent experiments in connection with those made since 1879 a very instructive comparison of the development of the German cement industry with reference to the strength of the product, particularly in tension and compression, is obtained. Mention has already been made of the improvement in regard to fineness. While the tests for compression were only occasionally made in 1883, they have become, since 1887, the most important factor for judging the qualities of cements.

Table A shows that cements having a tensile strength of more than 15 kg. per square centimeter (215 lbs. per square inch) form the greater part of those tested. The percentage of such for the different years was:

In 1880	68.2 %	In 1886	80.7 %
" 1881	94.7 "	" 1887	80.9 "
" 1882	88.3 "	" 1888	73.8 "
" 1883	59.7 "	" 1889	77.4 "
" 1884	84.8 "	" 1890	72.7 "
" 1885	74.2 "	" 1891	85.7 "

It is seen that from 1886 to 1890 there was no increase, perhaps because during this period the effort was made to reach the specified compressive strength of 160 kg. per square centimeter (2 300 lbs. per square inch) with a tensile strength of 16 kg. per square centimeter. But if the cements between 15 and 16 kg. per square centimeter in tensile strength should be counted, probably the years 1888–90 would not have lower percentages than those preceding.

Results of Tests of Portland Cements Made in the Royal Testing Station for Building Materials at Berlin.

A. Tensile Strength According to Standards of November 10th, 1878.

Year of tests.	Number of 28-day tests.	Tensile strength in kilograms per square centimeter.									
		Below 10.		Between—						Above 30.	
				10 and 15.		15 and 20.		20 and 30.			
		Number.	Percent.	Number.	Percent.	Number.	Percent.	Number.	Percent.	Number.	Percent.
1879-80...	22	...	....	7	31.8	1	4.6	13	59.1	1	4.6
1880-81...	38	1	2.6	1	2.6	13	34.2	19	50.0	4	10.5
1881-82...	77	3	3.9	6	7.8	39	50.7	25	32.5	4	5.2
1882-83 ..	57	5	8.8	18	31.6	21	36.8	11	19.3	2	3.5
1883-84...	79	2	2.5	10	12.7	27	34.2	39	49.4	1	1.3
1884-85...	89	2	2.3	21	23.6	33	37.1	33	37.1	...	....
1885-86 ..	109	2	1.8	18	16.5	37	33.9	41	37.6	10	9.2
1886-87...	68	...	....	12	17.7	20	29.4	31	45.6	4	5.9

B. Tensile Strength According to Standards of July 28th, 1887.

Year of tests.	Number of 28-day tests.	Tensile strength in kilograms per square centimeter.					
		Below 16.		Between 16 and 20.		Above 20.	
		Number.	Percent.	Number.	Percent.	Number.	Percent.
1887-88.....	103	29	28.2	32	33.0	42	40.8
1888-89.....	137	31	22.6	59	43.1	47	34.3
1889-90.....	99	27	27.3	38	38.4	34	34.3
1890-91.....,....	112	16	14.3	43	38.4	53	47.3

C. Compressive Strength According to Standards of July 28th, 1887.

Year of tests.	Number of 28-day tests.	Below 160 kg. per square centimeter.		Between 160 and 200 kg. per square centimeter.		Above 200 kg. per square centimeter.	
		Number.	Percent.	Number.	Percent.	Number.	Percent.
1883-84.....	21	15	71.4	5	23.8	1	4.8
1884-85.....	29	12	41.4	7	24.1	10	34 5
1885-86.....	26	9	34.6	4	15.4	13	50.0
1886 87.....	22	7	31.8	6	27.3	9	40.9
1887-88.....	53	30	56 6	13	24.6	10	18.9
1888-89.....,....	121	50	41.3	38	31.4	33	27.3
1889-90.....	84	16	19.0	25	29.8	43	51.2
1890-91.....	85	28	33.0	26	30.6	31	36.5

The endeavor to secure a high compressive strength is plainly seen in the last table, the number of those below the standard continually decreasing, while those between 160 and 200 kg. per square centimeter (2 300 and 2 900 lbs. per square inch) slightly increase, and those above 200 kg. per square centimeter materially increase, in number.

If we cast a retrospective glance over what has been already said regarding the standard methods of testing of Portland cements, it will be apparent that impartial testing is not an easy thing, and that a certain training is necessary in order to observe the different regulations and secure trustworthy results. Accordingly, at the request of the Association of German Portland Cement Manufacturers, the Royal Testing Station for Building Materials at Charlottenburg, near Berlin, was designated by the Minister of Public Works of Prussia, by a decree of August 16th, 1880, as the authority for settling technical questions in dispute. At the same time it was ordered that in contentions between sellers of cement and the Bureau of Public Works concerning the quality of cement, the decision of this testing station, made after proper tests, should be final and binding upon both parties with respect to all technical points.

Although the standard tests are entirely satisfactory for these purposes and for the comparison of specimens, yet they are not sufficient to completely define all the properties of a good Portland cement, and its special qualities for special purposes. Hence, in Germany, methods of testing—apart from the standards—have become further developed, and there are several in use which cannot be passed over in silence, since the international commission above mentioned has approved and recommended them.

Weight may be mentioned as an important distinctive element. To determine this, cement may be allowed to fall from a sieve into a liter measure, which is then shaken. The apparatus of Tetmajer was recommended for this purpose by the commission of September 20th, 1890, but it has not yet come into use in Germany. The determination of the specific gravity of the cement particles by the volume-meter of Schumann is a well-known uniform method. This consists of a glass bottle of about 200 cu. cm. (12.2 cu. ins.) capacity, with a calibrated glass tube in its neck. The bottle is nearly filled with oil of turpentine, the tube tightly inserted and filled by a pipette with the same oil to the zero mark of the scale, care being taken that all air bubbles

are removed. One hundred gr. (3.5 ozs.) of cement is put in through the tube, which is then closed by a cork. When the fluid becomes clear, the height of its top surface is noted on the scale. The weight of the cement divided by its volume, as determined by the scale of readings, gives the specific gravity. To secure precise results, it is necessary that the temperature should remain uniform throughout the experiment, and hence vessels, cement and oil must have been kept in the same room for some considerable time. In hot weather the apparatus can be put into water of a known constant temperature. If 100 gr. of cement are used, a rise of 1° Cent. between the two readings decreases the specific gravity 0.8 per cent.

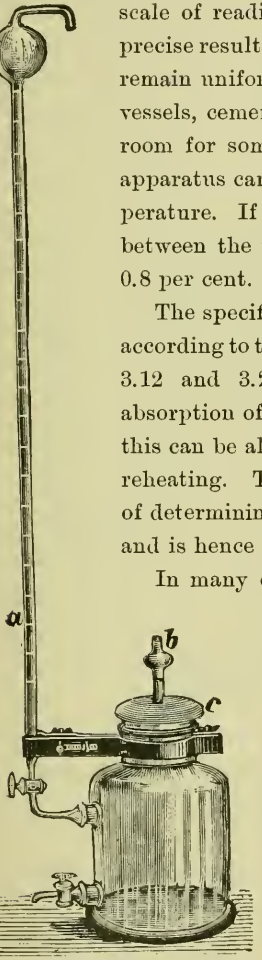


FIG. 13.

The specific gravity of fresh Portland cement varies, according to the brand and the proportion of lime, between 3.12 and 3.25. Stored cement becomes heavier by absorption of water and carbonic acid from the air; but this can be almost restored to the original condition by reheating. The specific gravity affords the only means of determining the calcination of a cement with certainty, and is hence of high value.

In many cases it is of importance to know whether the briquettes tested for tension and compression were prepared of uniform density, and this can only be ascertained through the specific gravity. For this purpose Seger's volume-meter can be used. This is a glass vessel with two tubes inserted in one side, provided with a cap *c*, which is funnel-shaped on top, so that no air bubbles can long be retained, and which has a tube *b*, where a zero point is marked. The tube *a* is provided with an outlet cock below and with a ball on top of about 100 cu. cm. (6.1 cu. ins.) capacity. The lowest tube is merely an outlet cock. The apparatus is filled with water, air bubbles being avoided, up to the zero mark on both the tubes *a* and *b*. Then the water is drawn up into the ball of the tube, the outlet cock opened, the speci-

men placed in the water, and the water again allowed to rise to the zero mark. The height of water in the tube then shows the volume of the briquette, and the specific gravity is the ratio of its weight to its volume. Finally, there is one more method of testing to be mentioned which has been repeatedly used at the Berlin Testing Station, as well as by Professor Bauschinger, of Munich, and which seems capable of giving valuable information regarding the binding properties of cement. This is the test for wearing qualities of cement (Fig. 14).

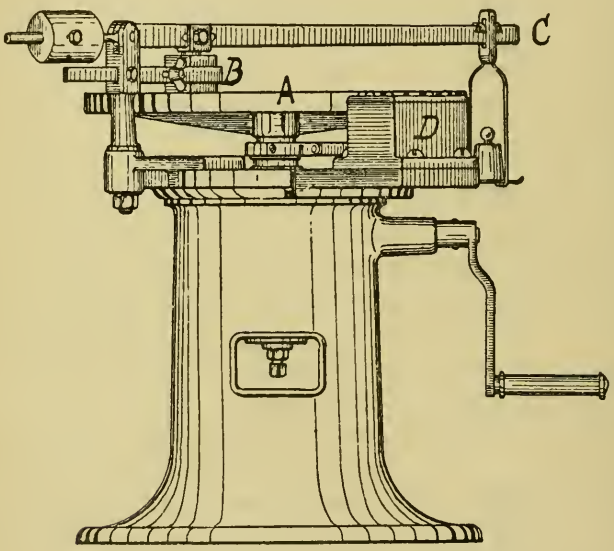


Fig. 14.

The method used at Berlin consists in pressing the body to be tested (*B*) by means of a simple lever loaded with 25 kg. (56 lbs.) against a cast-iron disc rotating at the rate of about 22 revolutions per minute; 20 gr. (308 grains) of Napos Quartz No. 3 are put on the plate at the start, and a like amount at the end of every 15th turn. After 30 revolutions of the disc the body, which was weighed before starting, is again weighed and the loss determined. The revolutions are automatically registered at *D*.

When making the tests, cubes 1 cm. in size are made with the regulation hammer apparatus, after the usual manner of making specimens to be tested for compression. After hardening 7 or 28 days in air or under

water they are weighed. By means of Seger's volume-meter, the density is next determined and the body then subjected to the above-described friction process. The quantity lost in this test is shown by the expression $\frac{g - g_1}{g : v} = v\ 7$ (for a seven days' specimen, in which g and g_1 are the first and last weights and v the volume. The same specimens are used for the 28 days' tests, but the density must be determined anew just before beginning the test, since the weight of the specimens alters under the influence of air and water. The density after 28 days is $v\ 28 = \frac{g\ 28}{v_1}$ in which $g\ 28$ is the weight of the specimen just before beginning the 28 days' test, and $v_1 = V - v\ 7$, or the original volume V diminished by the amount in cubic centimeters lost in seven days ($v\ 7$).

For the 28 days' test divide the weight lost by friction by $V\ 28$, to obtain the quantity $v\ 28$.

In order to compare the amount of loss sustained by different cements and mortars, the results of experiments made at six different cement factories are given below. Cement No. 1 is a Holstein brand; Nos. 2, 3 and 6, Silesia, and Nos. 4 and 5, puzzuolana cement. While the last-mentioned cannot properly be compared with Portland cement, they are inserted here, to show the marked difference of the behavior of the two varieties. All these cements fulfill the Prussian regulations. They have a tensile strength, after 28 days, when used in 1:3 mortar, of more than 16 kg. per square centimeter (230 lbs. per square inch) and a compressive strength of more than 160 kg. per square centimeter (2 300 lbs. per square inch). The sieve of 900 meshes rejects less than 10%, and they are of constant volume. The table shows the quantities lost by wear of these six brands, in cubic centimeters, for specimens which hardened in damp rooms in the air, and also for specimens which were kept in air for 24 hours, and the remainder of the time under water, being taken out of the water one hour before abrasion.

For all these cements the loss of weight of the air specimens is greater than that of the water specimens. The loss of weight of the seven-day specimens is only slightly greater, indeed less in some cases, than for the 28-day specimens. This striking phenomenon is explained by the circumstance that after hardening for seven days the specimen has a certain toughness, it being in the first stage of

development; after 28 days, however, it has acquired a slight brittleness, which renders it more susceptible to abrasion and this it

TABLE SHOWING LOSS OF WEIGHT OF CEMENTS BY ABRASION.

Number.	Length of tests.	Hardened in Air.					Hardened under Water.				
		Neat cement.	1 cement. 1 sand.	1 cement. 2 sand.	1 cement. 3 sand.	1 cement. 4 sand.	Neat cement.	1 cement. 1 sand.	1 cement. 2 sand.	1 cement 3 sand.	1 cement. 4 sand.
1.....	7 days.....	5.5	4.2	3.5	6.1	8.9	5.0	3.7	3.8	7.9	10.4
	28 days.....	7.9	4.5	1.9	9.7	13.8	2.9	1.8	2.7	4.1	6.8
2.....	7 days.....	4.6	2.5	3.4	8.3	9.5	4.6	1.7	3.0	6.5	8.8
	28 days.....	3.9	2.0	3.5	6.0	7.3	3.6	1.8	2.3	5.0	5.6
3.....	7 days.....	5.4	2.9	6.1	18.3	62.8	2.4	2.1	2.1	4.7	15.9
	28 days.....	2.4	1.6	1.9	2.3	22.9	2.2	1.3	1.4	1.8	2.2
4.....	7 days.....	10.4	3.9	3.9	7.3	17.3	4.0	2.6	2.5	3.6	4.6
	28 days.....	9.0	4.2	3.4	5.2	14.9	3.6	2.8	2.1	2.1	1.8
5.....	7 days.....	10.6	7.6	10.2	19.2	28.7	7.0	5.9	7.1	15.7	17.4
	28 days.....	5.3	1.6	1.6	9.0	11.6	5.8	0.9	1.1	2.3	13.4
6.....	7 days.....	6.2	2.3	4.2	6.5	17.1	3.5	1.3	2.2	5.0	10.9
	28 days.....	5.3	2.2	3.3	4.4	8.9	3.9	1.3	2.2	2.7	3.9

practically always retains. The air specimens also possess a greater brittleness than the water specimens. For both kinds the table shows that the loss of weight of the neat cements is greater than that of mortars until the dose of sand becomes very high. Hence, the higher the proportion of sand can be made, until the loss of weight of a mortar exceeds that of a neat cement, the better will it be for floors and pavements, as far as wearing qualities are concerned.

By the grinding action of the apparatus small particles are torn off the specimen. These particles are fine flour of cement for neat cement specimens; for a specimen having 1 part sand to 1 of cement this fine flour cannot be formed from the cement between the grains of sand as long as there is sufficient binding force to hold the grains together to resist the tangential action of the grinding plate. This binding force of the cement must, of course, be less the less the proportion of neat cement in the specimen and the greater the dose of sand. One cement will possess more, another less, of this binding force, and can hence bear a greater or less proportion of sand with an equal loss of weight. Furthermore, it will be readily seen that the binding strength of a cement is increased by the power of the cement grains to adjust themselves close to the quartz grains; in other words,

by the fineness of the cement. In fact, we see from the table that the very fine cements, to which class belong the puzzuolanas, possess the greatest cohesive strength, entirely independent of their remaining properties. This may be more plainly seen if we select the 28 days' wearing tests and show them for air and water mortar in a graphic manner. The doubly hatched areas in Fig. 15 show the mortars which hardened under water.

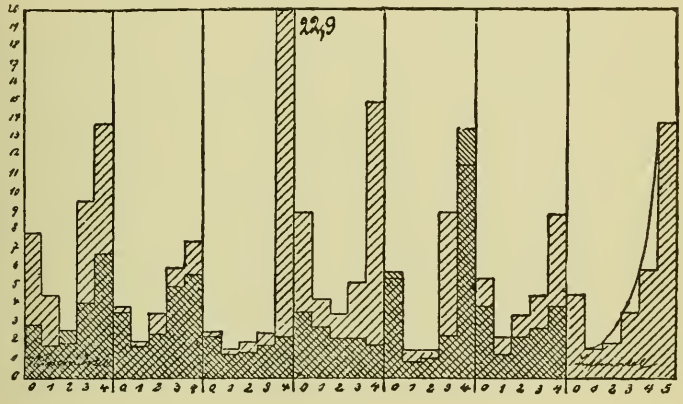


FIG. 15.

The abscissas are arbitrarily assumed and the co-ordinates show the wear in cubic centimeters of various Prussian regulation cements mixed in the proportions of 0, 1, 2, 3 and 4. A great similarity is shown by the figure in the wearing qualities of the water specimens in cements Nos. 1, 2 and 6. The decrease in binding strength is very nearly proportionate to the amount of sand added. No. 3 shows (on account of its extraordinary fineness) a strikingly low amount of wear which continues for hydraulic mortar up to an addition of from 3 to 4 parts of sand, while air mortar loses its bond in a marked degree with 4 parts of sand. The two puzzuolanas show a wide variation in the amount of wear, which amount, although especially in the case of the hydraulic mortars, is not great, nevertheless makes them compare unfavorably with the Portland cements. For the sake of comparison a representation of the relative wearing qualities has been added to the figure. This is the mean of tests made by Professor Böhme on samples furnished by 28 different factories. The figure shows plainly in what manner the wear increases with the increased amount of sand.

The least loss by wear is found in 1 part cement to 1 part sand. In the mean the amount lost by wear is:

For	neat Portland cement				.....	4.7	cu. cm.
"	1 cement, 1 sand				.....	1.7	"
"	1	"	2	"	.....	1.9	"
"	1	"	3	"	.....	3.6	"
"	1	"	4	"	.....	5.9	"
"	1	"	5	"	.....	13.8	"

For comparison other abrasion tests are given as follows: plaster of paris, Rabitz patent, 5.5 cu. cm.; slag plates, 2.8 cu. cm.; sandstone, 1.7 to 15.0 cu. cm. Much additional data might be furnished showing the importance of tests for abrasion, but the limited space forbids.

In conclusion, two desirable tests remain to be mentioned which have not come into general use because reliable and convenient apparatus has been wanting. These are the test for adhesion of the binding materials and the tests for imperviousness to water. The Berlin conference of 1890 left the question regarding the adhesive strength of hydraulic binding materials unsettled, but it is to be hoped that the near future will, thanks to the unremitting labors of individual members of the Commission, bring a satisfactory solution. The history of the Association of German Portland Cement Manufacturers is a guarantee that it will not cease pushing its scientific investigations regarding the properties of its product, thus conferring blessings upon the German Portland cement industry and benefits upon architects and engineers over the entire earth.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

634.

(Vol. XXX.—October, 1893.)

ON THE MANUFACTURE AND TESTING OF PORTLAND CEMENT.

By HENRY FAIJA, M. Inst. C. E.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

Portland cement, as is well known, consists of a chemical combination of lime, silica, and alumina with iron in certain well-defined proportions, together with alkalies, magnesia, etc., which enter, in a minor and less essential degree, into its composition.

The lime may vary from..... .....58 to 64%
The silica " " .....18 " 24 "
The alumina and iron from..... 8 " 14 "

the three together amounting to about 95 or 96% of the whole, and the properties of the cement produced will depend, in the first instance, on the proportions which these ingredients bear to one another. The magnesia, alkalies and sulphuric anhydride, which are found in

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

small varying quantities in nearly every cement, affect its properties in a minor degree, in accordance with the proportions which they bear to each other, and to the whole.

The difference between a hydraulic lime and a Portland cement is that in its composition hydraulic lime contains a larger percentage of lime than a Portland cement, and consequently its nature is such that it requires hydrating before use, whether that hydration be effected by grinding or slacking it with water, whereas Portland cement is fit for use without previous hydration ; this difference being due not only to the difference in composition, but also to the degree to which calcination is carried.

Hydraulic lime is the product of the simple calcination of a limestone rock, and any slight variations in the composition of the natural rock does not materially affect the resulting lime ; but the composition of a Portland cement has to be kept within certain well-defined and comparatively close limits, and consequently a natural rock is seldom found which is capable by simple calcination of producing a Portland cement, and it is therefore necessary to find such limestones and clays or shales, which, by their admixture in the proper proportions, will produce a material having the desired composition.

The processes of manufacturing Portland cement, therefore, consist in the first instance of obtaining a perfect mechanical admixture of such raw materials as are suitable and available, and subsequently reducing this mechanical admixture to a chemical one by calcination, and afterwards grinding the clinker so produced.

Given any materials which contain among them the lime, silica and alumina, and which may, by proper admixture in definite proportions, produce the components of a Portland cement, then Portland cement may be produced from them; but it is needless to say that the commercial aspect of the undertaking should be considered, and in many cases it is evident that the cost of production would be prohibitive. It is therefore necessary, in selecting the raw materials, to select those which are the most amenable to reduction and admixture.

Those materials which are capable of reduction by water are not only the most economical to reduce, but are also capable of producing the most perfect mechanical admixture, and it is needless to say that the more perfect the mechanical admixture of the several ingredients, the more perfect is the chemical compound produced by calcination.

The chalks and clays from which Portland cement is produced on the Thames and Medway, in England, and the chalk and clay which are used at Yankton, Dakota, U. S. A., at the works of the Western Portland Cement Company, which the author designed for the proprietors some four years ago, are examples of materials which lend themselves easily to disintegration and reduction by water.

Other raw materials which are not capable of reduction by water have to be ground to powder and then mixed, and it is evident that such a process must be more expensive than the former ; still, by the adoption of reducing machinery, suitable to the materials to be treated, the extra expense need not be great. Examples of such materials are to be found in the blue lias formation from which many cement works in the Midland Counties of England derive their supply of raw material. In this case, the stone, and the shale which separates the beds of stone, are both used in the production of cement, the proportion of shale to stone used being determinen by the chemical composition of each; but as both are capable of being ground as brought from the quarry, they are generally ground in their proper proportions together, the grinding and subsequent pugging effecting the mixing.

Another modification in the means of obtaining a mechanical admixture of two materials, is where one is capable of being reduced by water and the other is not ; as, for instance, a hard limestone and a clay. Such an instance, among many others, is found at Parahyba do Norte, Brazil, at works which the author designed three years ago. In this case a hard limestone is used in conjunction with a river mud, and the author found that the best means of producing a thorough admixture of the materials, was by grinding the limestone, drying and then grinding the mud, and then weighing and mixing them in their proper proportions.

It would be quite impossible to enumerate the several mechanical devices which have to be used for the economical production of a perfect mechanical admixture of the different materials. It is necessary to consider the physical properties of the materials to be dealt with, and to treat them in such manner as experience may suggest to be the most satisfactory, combined with due economy.

Having so far considered the method of preliminary dealing with the raw materials, the next process is to make them into such form that they are capable of being loaded into a kiln for calcination.

For the further treatment of materials which are reduced and mixed by water, there are two well-known processes, both processes being subject to many variations in detail. The first, and the oldest, and probably, all things considered, the most satisfactory, except in the matter of economy, is to wash the chalk and clay together in a wash-mill with a very large quantity of water. The slip so produced runs away by an overflow, and is conducted by proper channels into large reservoirs or backs, where the solid particles in the slip gradually sink to the bottom, and the clear water is drawn off by weirs and sluices. When a back or reservoir is full, and the slip has attained sufficient solidity to be removed in barrows, it is laid on the drying floor, and subsequently loaded into the kiln. This process, it will be readily understood, produces the most perfect mechanical admixture of the two materials which it is possible to attain.

The second well-known method is to wash the chalk and clay in a wash-mill as before, but with considerably less water, and instead of taking the overflow, the slip produced is simply allowed to pass through a grating, and, consequently, it is not reduced in the wash-mill to that same degree of fineness to which it is in the previously-described process. In order, therefore, to obtain that required fineness, the slip, as it passes through the grids of the wash-mill, is conducted to a pair of ordinary millstones and ground, from whence it passes direct on to the drying floors, and is dried ready for calcination.

As will be seen, these two processes lend themselves to many modifications, in accordance with exigencies of the site, etc., and the fancy or whim of the manufacturer.

When dealing with materials which are reduced by grinding, there are two well-known methods. The one is to mix it in a pug mill with a fairly large quantity of water, reducing it into a kind of stiff slip, which may be dried on a drying floor; the other is to mix it with a small quantity of water and make it into bricks in any of the well-known dry brick machines, which bricks are then dried in the ordinary way previous to calcination.

The difference in the two processes does not seem very great, but there is perhaps more difference between them than is apparent at first sight. When mixed into a stiff slip, there is possibly from 25 to 30% of water to evaporate from it before it is sufficiently dry to load into the kiln, at the same time the dried slip has the advantage of being more

or less porous, and consequently is fairly easy to calcine. In the process of compressing the slip into dry bricks, or briquettes, the amount of water to evaporate is perhaps only 12 or 13%, but owing to the density of the brick it is difficult to evaporate this water, and the dried brick does not lend itself readily to easy and regular calcination in the kiln.

The means adopted for drying the slip, slurry, or bricks, are various. The oldest and perhaps best known is that of having ordinary tiled floors with flues underneath, through which the combustion from coking ovens is passed, the slip being laid on the top of the tiles ; by this means the the slip is dried, and a certain quantity of coke is produced, which is used as a supplementary supply to that which is required for the calcination of the cement in the kilns. Other floors are of iron plates, under which free steam is passed, and again there are many drying floors which are constructed like ordinary brick-drying floors, without any pretention to economy. But the most advanced method is to construct the drying floor at the level of the top of the kiln, and to use the waste heat from the kiln for drying the slurry or bricks for the next loading. Kilns or drying floors on this principle have been the subject of innumerable patents, both in the United States and Great Britain, and it would be impossible to even enumerate them, to say nothing of considering their respective merits. It is only possible to say, that the primary object of a kiln is to calcine the cement evenly and regularly, and that the drying of the slip by the waste heat from it must be subservient to the proper calcination of the cement.

So far, the preliminary treatment of the raw materials, their proper and thorough admixture, and their manufacture into a suitable form fit for calcination, have only been considered, and it is needless to say, that no matter how perfectly the manufacture has been so far carried, the final result will not be satisfactory unless the calcination is perfect. The requirements of calcination are : that the calcination should be stopped just short of vitrifaction ; that the proper degree of calcination should be effected rapidly, and that the clinker should be burned, not baked ; that the product of a kiln should show an even and regular degree of calcination throughout ; and lastly, that these results should be obtained with due economy with respect to fuel ; and the kiln which best satisfies these requirements is the one to be adopted.

At the same time it must not be forgotten that although the ingredients of a Portland cement are the same, no matter where, or from what raw materials the cement is made, and that the chemical combination of those ingredients should be carried to the same degree of firmness, still different materials require different degrees of calcination, and that a kiln which is eminently suited for one slip is not of necessity suitable for the calcination of a slip produced from other raw materials. The whole duty of a kiln is first to expel the carbonic acid, and what remains of moisture, from the slip, and then to create an intimate chemical combination of the ingredients.

On drawing a kiln, all light burnt portions should be picked out, and only the thoroughly burnt clinker passed to the crushers for subsequent grinding; and it is usual to put the light burnt on top of another kiln for further calcination, or in some works small subsidiary kilns are used for its further calcination.

The calcination of a Portland cement has hitherto been carried out in intermittent kilns, *i, e.*, a kiln is loaded, fired, allowed to cool, and drawn, and hence a great loss of heat and considerable wear and tear caused on the kiln, through alternate expansion and contraction; but the difficulty of altering this and adopting a continuous or running kiln has been the difficulty of obtaining a sufficiently refractory material to form the lining of the kiln; not so much on account of the heat in a cement kiln being greater than that to which fire bricks are subjected in many other manufactures, but to the presence of the lime in the cement acting on the silica and alumina in the bricks, causing them to flux and enter into combination with the slip of the cement.

The economical grinding of cement has lately attracted the attention of a great number of inventors, and mills or grinding machines of almost every conceivable design and principle have been patented; but whether any of these will survive and eventually supersede millstones is a very problematical matter. The two principles which have perhaps attracted the greatest attention are those of edge-runners and ball mills, and the economy in power by both these principles over ordinary millstones is very considerable, and the cost of repairs and maintenance is also, in most cases, considerably reduced; but whether the grinding is as efficient is another question altogether. Mere fineness does not satisfy the question, as a cement may be ground to an equal fineness in two different mills, and yet one will be all grit and

the other all flour ; and the more floury nature a cement is, the better will be the results obtained with it, both in the testing-room and in actual practice ; and undoubtedly no grinding machine that has as yet been invented will produce the same percentage of flour on equal grinding as the ordinary millstone. Mills on the ball principle give better results than those on the edge-runner principle, but are not so efficient as millstones.

The power consumed by the several principles, reduced to the production of 1 ton of cement per hour, may be approximately stated to be as follows :

For millstones.....	30 to 32 I. H. P.	per ton per hour
Ball principle.....	16 to 18 I. H. P.	" "
Edge-runner principle.....	12 to 14 I. H. P.	" "

in each case the cement being ground to a fineness of about 5% residue on a 50×50 sieve, and it will thus be seen that the power required is proportionate to the amount of flour produced.

The great objection to millstones, from a manufacturer's point of view, is the great expense entailed in dressing them, as in a hurst of four pairs of stones, one pair will always have to be up being dressed; and there is, therefore, not only the expense of dressing, but there is the increased capital charge in requiring four mills to do the work of three. It seems possible, though the author has not had the opportunity of trying it, that by giving the millstones a fine dress with a considerable depth of face, the first grinding of the cement might be effected in one or other of the grinding machines and finished only in the millstones.

Before leaving the subject of the manufacture of cement, the author would like to point out to manufacturers, or intending manufacturers, the necessity and great advantages of having ample warehouse room for the finished cement. Very few cements are fit for use immediately they are ground, and all cements are improved by judicious and careful warehousing. Not only therefore is a manufacture improved by having ample warehouse room, but ample warehouse room enables the manufacturer to continue manufacturing his full output, even when his sales may for the moment be a little slack.

The object of testing cement is to obtain a knowledge of the material which is about to be used; and the author maintains that that

knowledge can best be obtained by gauging a cement by itself with the addition only of water, and without the addition of sand or other materials, as these themselves, by variations in their composition, form and nature, introduce an element of error, independently of any good or bad qualities in the cement; also, that the results obtained in the testing-room and laboratory are infinitely superior, so far as strength is concerned, to any that can be obtained in actual practice; and that it is the object of the manipulator in the testing-room, to obtain the very best results which his knowledge and skill make possible.

A very true knowledge of the value of a cement may be obtained by determining the following properties:

1. The time which a cement takes to set.
2. Its soundness or freedom from blowing.
3. The fineness to which it is ground.
4. Its tensile strength at three or seven days.

Of these the most important is, of course, its soundness, for if a cement will, after a longer or a shorter period, "blow" and destroy the work of which it forms a component, it evidently does not matter how long it takes to set, to what fineness it is ground, or what tensile strength it develops within the limits of time of an ordinary test.

The manner of carrying out this test will be considered later on.

The setting properties of a cement may be determined by taking a few ounces of the cement and mixing it with the very smallest quantity of water which will allow of its being worked up with a trowel into a tenacious mass, which will retain the form into which it is made. This should be shaped up into a pat of about 3 ins. long, by $1\frac{1}{2}$ ins. wide, and $\frac{3}{4}$ in. thick, and placed on a non-porous slab; and it may be considered "set hard" when the pressure of the thumb nail will no longer mark it. Or if the thumb-nail test is considered too primitive, a Vicat needle may be used; the needle having a flat point with a diameter of 0.1 in., and loaded with a weight of 3 lbs. The needle should be allowed to remain on the pat for about one minute, and when on removal no mark is left, the cement may be considered set hard. There is, however, an intermediate stage which should be noted, viz., the time of "initial set," and this is of more importance than the time which a cement takes to finally set. A cement when it is first worked up into a pat, has a glossy, wet appearance,

which is due to the excess of water which was used in gauging remaining on the surface, and it will remain in this condition until the setting commences; the water will then leave the surface, and become absorbed by the cement as it sets or crystallizes, and when once this process has commenced, any disturbance of the cement will destroy its constructive value; hence it will be seen that the time of "initial set" of a cement, or the time that elapses between the first addition of water and the commencement of setting, defines the time which may be taken for the proper manipulation of the cement in the work in which it is to be used. The time which it takes for a cement to set hard is not of such importance, and also cannot be determined with the same accuracy. The initial set is the commencement of an actual chemical process which continues until the cement has attained its ultimate hardness and strength, while the time of "set hard" forms only a somewhat more or less accurately to be determined period. At the same time the comparisons between the time of "initial set" and "set hard" afford many opportunities of obtaining considerable knowledge of the nature of a cement, and the manner in which it may best be used.

Cements may be divided into two classes—quick setting and slow setting. A quick setting cement may have an "initial set" of seven or eight minutes, and will be "set hard" within the hour; in a slow setting cement the time of initial set is sometimes very difficult to define, but it may be set hard in from two to six hours.

The comparative constructive values of quick or slow setting cements must depend entirely on the work for which the cement is to be used. It cannot be said that a quick setting cement is better than a slow setting cement, or a slow setting cement better than a quick setting cement; they may be equally good cements, though not adapted to the same purpose. There are, of course, some cements which do not arrive at the point which may be considered "set hard" in less than 24 hours; there are also some of the quick setting cements which become set hard almost immediately on the addition of water, rendering it absolutely impossible to gauge them. Either of these extremes are undesirable, and a cement that is abnormally quick setting or abnormally slow setting, should not be used in any work until after very careful tests and examinations have been made to otherwise determine their quality. There are some cements which have a very quick

"initial set," but after that take some considerable time to harden; others again, though really quick setting cements, approach to the characteristics of slow setting ones by having no "initial set" which can be readily defined; all these may be perfectly good cements, but as they possess abnormal characteristics, should be very carefully examined before use.

The causes which affect the setting of a cement are: primarily, the proportions to each other of the materials of which it is composed; and secondly, the degree of their chemical affinity, or in other words, the degree or manner of calcination to which they have been subjected.

In former days a very slow setting cement was supposed to be so, because it contained a very large percentage of lime. This is true only so far that a cement containing a large percentage of lime will probably be slow setting, but the slow setting nature of a cement may be due to many other causes, and the most marked of these is the degree of calcination to which it has been subjected. Given any combination of lime, silica and alumina, which falls within the limits of an ordinary Portland cement compound, the degree to which it is calcined will make it, within limits, dependent on its composition, either a quick or a slow setting cement. The percentage of alumina and iron again will affect the setting of a cement to a great extent.

Independently, however, of the chemical composition and calcination of a cement, there are other matters, of a purely mechanical nature, which affect the setting powers of a cement. The age of a cement is perhaps the most important, and there are one or two peculiarities in this; a cement when first ground may generally be gauged very easily with a comparatively small quantity of water, but when that cement has been in the warehouse for 24 hours it may be almost impossible to gauge it, as it sets or commences to set almost directly the water is added; after this period, however, the cement gradually becomes slower setting, and even a very quick setting cement will in a few months become quite slow enough for all ordinary purposes, provided, of course, that during that period it has been kept in a dry and cool warehouse, and has not been affected by damp or wet, or otherwise damaged.

The tensile strength of a cement is the test which is generally considered to most accurately define its value for constructive purposes;

needless to say the strength of a cement will be dependent on its composition and calcination in a similar manner to the peculiarities of setting which have already been referred to. A quick setting cement naturally attains greater strength in a shorter period than a slower setting one, but a slow setting cement has probably the greater ultimate strength.

The manner of carrying out this test is to gauge briquettes in gun-metal moulds, having a sectional area at the smallest part of 1 sq. in.; the briquettes after being left in the moulds for 24 hours in order to become perfectly set, are removed and placed in tanks of water, in which they are allowed to remain until they are to be tested. The usual and most convenient periods to test the briquettes for tensile strength, is at the expiration of three and seven days from the time of gauging. It is usual to make five briquettes to test at each date, and the average strength of the five taken as representing the tensile strength of the cement at those periods. It is also perhaps desirable to occasionally make briquettes to test at a longer date, usually 28 days, as by that means a corroboration of the opinion formed of the cement at the expiration of the seven days' test may be obtained. It may not perhaps be apparent that it is absolutely necessary to make tests at two or more dates in order that the value of the cement may be arrived at, but it will be readily seen that the increase in strength between the several dates of testing gives an indication of its growth, and hence an approximation of its ultimate strength may be arrived at.

It is difficult to define any hard and fast rule which should govern the increase in the strength of a cement between the three and seven days' test. Many quick setting cements will carry a tensile strain of 400 lbs. on the inch section at the expiration of three days from gauging, and will probably carry 500 lbs. at the expiration of seven days; this would show an increase of 25%, and is perhaps as much as can be expected from a cement which develops a very high tensile strength at the early date. A slow setting cement will probably at the three days' test not carry more than 300 lbs. on the square inch, and perhaps 450 lbs. at the expiration of seven days from gauging, which would represent an increase in tensile strength of about 50 per cent. It is also known from experience that most slow setting cements will continue to increase in strength for a much longer period than the

quicker setting ones, and consequently the slower setting cements, under ordinary circumstances, will attain a greater ultimate strength.

It will therefore be seen that if a slow setting cement is required, it is not advisable to demand, in a specification, too high a tensile strain at the early dates, and possibly 350 lbs. on the square inch at the seven days' test is sufficient, whereas if a quick setting cement is required, 400 lbs. is nothing too much to demand. Of course there are variations from this regular order of things, and sometimes a slow setting cement may be found that carries 400 lbs. or 500 lbs. at the expiration of the three days' test, also there are quick setting cements that have a comparative low tensile strength at the early dates, but give indications of considerable growing powers ; and it is only possible to say that any cements that develop abnormal strength, whether in excess or deficiency, should be looked upon with a certain amount of suspicion until their properties and qualities are fully ascertained.

Latterly, the author has had several specifications before him, which, in addition to naming the minimum strength at each date, have also defined the maximum. This, no doubt, has been devised with the object of securing a certain good increase in strength between the several dates at which the cement is tested, but it seems almost needless to point out that such a specification defeats its own object, for whereas the best results of a cement can only be obtained by careful and proper manipulation in the testing-room, a lower result may be easily secured by indifferent or careless gauging.

The sand test consists in gauging the cement with 3 parts of sand, which should be of approved quality, sifted to a certain size and properly washed and cleansed, but the difficulties of carrying out the test are many. Variations in the form and hardness of the grain of sand materially affect the result of the test, and the difficulties of manipulation and of making solid briquettes render it altogether an undesirable test to adopt, irrespective of which the test is a long one, the briquettes not being tested until 28 days after gauging, and it is needless to say that in very many cases it would be impossible to wait that length of time to know the value of the material which it is required to use. In the author's opinion cement should be tested by itself, not only because the manipulation is considerably simpler, but because it is unwise to introduce into a test extraneous matters and complications which are in themselves open to considerable variation.

If it is desired to ascertain the strength of a mortar compounded with any particular cement, then let the cement be gauged with those aggregates and sand which are to be used on the work ; by this means some definite information may be obtained as to the strength and binding power of the mortar which is to be used ; but to test a cement with what it is pleased to call a normal or standard sand, gives practically no information in this direction, and simply tends to complicate and confuse an otherwise simple test.

The fineness to which a cement is ground materially affects its constructive value, but it is unnecessary to say much about it, as the principle of fine grinding is perfectly well understood. Probably, if a cement was ground to an impalpable powder, the best results would be obtained; but, as it is impossible for the manufacturer to produce this degree of fineness with the machinery at his command, except at great cost, it is not desirable to demand such extremely fine grinding. A cement that will all pass through a sieve having 625 holes (25^2) to the square inch, and which will leave a residue of from 5 to 8% when sifted through a sieve having 2,500 holes (50^2) to the square inch, is for all practical constructional purposes ground fine enough.

It is now necessary to refer to the property which it is essential that all cements should possess, viz., absolute freedom from all indications of either expansion or contraction, and that when once set it shall in no way alter its form, crack, or disintegrate.

One of the peculiarities of Portland cement is, that if its components are improperly proportioned, or its manufacture has not been properly carried out, it may have a tendency, after being gauged and mixed with water, to crack, expand, or disintegrate and fall into powder. This peculiarity is known under the cognomen of "blowing," and when a cement is said to "blow" or to be a "blowey cement," it means that after the cement has been used it expands, cracks or disintegrates, destroying the work in which it has been used.

The cracks, however, which are seen in concrete work are not always due to the use of a "blowey cement," but may be due to constructional causes, or to the expansion and contraction of the structure due to variations in temperature, or to the natural contraction of the mass ; and a simple crack in a piece of concrete would hardly be indicative of a "blowey cement" unless accompanied by other indications, such as friability or absolute disintegration.

Concrete or mortar, again, may disintegrate, crack and fall to pieces from other causes than the use of a "blowey cement." There are certain matters often present in aggregates which, by not allowing the cement to set properly, are antagonistic to the production of a sound concrete or mortar; the principal of these are dirt and loam, and there is no doubt that in numerous instances the cement has been blamed when the real fault has been either that the aggregate with which it was used was dirty or unsuitable, or that the concrete or mortar had been improperly manipulated ; and a user of cement should be as careful in his choice of aggregate, sand and water as he is in his choice of cement. The best aggregates are those which, while having ample strength, are somewhat irregular in form, and slightly porous, and which have been carefully and thoroughly washed before being used. The scope of this paper, however, does not extend to the choice of aggregates and manufacture of concrete, but as these, if improperly selected and manipulated, may cause a failure, irrespective of the quality of the cement, it seemed necessary to allude to the subject.

Returning, however, to the subject, a cement may blow within a few hours of its being gauged, or it may not blow until several months afterwards. A cement may blow when it is very fresh and newly ground, and will lose that tendency after it has become aged. Some cements will blow whether they are new or old.

The cause of "blowing" in a cement is generally due to an excess of lime in its composition, or to an imperfect combination of the lime with the silica and alumina. It may, however, be due to other causes; as, for instance, to the presence of other basic materials unduly entering into the composition of the cement by the use of improper raw materials. One of these, magnesia, created a considerable scare a few years ago. Sulphate of lime or gypsum is another, which, although it has not attracted the attention of users like magnesia, is more often found in cements, and, when in any considerable quantity, undoubtedly has a very great power of rendering a cement blowey. As previously stated, it is hardly fair to the user that he should be required to make himself *au fait* on the several causes which constitute a blowey cement ; it should be enough for him to determine, and be able to ascertain, whether or not a cement is blowey, and leave it to the manufacturer to properly compound the cement, and correct his manufacture.

Several means have from time to time been devised for ascertaining within the limits of time of an ordinary test whether or not a cement is absolutely sound, and that process or test which was devised by the author some 14 years ago is now in general use. The apparatus in which the test is carried out, and the means of carrying out the test, are fully described in the *Transactions* of the American Society of Civil Engineers, Volume xvii, November, 1887, in a paper which the author had the honor of communicating to that Society, headed "Portland Cement Testing." Briefly, it is a vessel containing water, the water being maintained at an even temperature of about 110 to 115° Fahr. ; there is a cover to the vessel, so that above the water there is a moist atmosphere which has a temperature of about 100° Fahr.

The manner of carrying out the test is by making a pat, in the manner already described, on a small piece of glass ; immediately the pat is gauged it is placed on a rack in the upper part of the vessel and is there acted upon by the warm vapour rising from the hot water, when the pat is set quite hard it is taken off the rack and put bodily into the water, which, as has already been stated, is maintained at a temperature of 110 to 115° Fahr., and in the course of 24 hours it is taken out and examined, and if found then to be quite hard and firmly attached to the glass, the cement may at once be pronounced sound and perfectly safe to use ; if, however, the pat has come off the glass and shows cracks or friability on the edges, or is much curved on the under side, it may at once be decided that the cement in its present condition is not fit for use. The blowing, however, may only be due to the extreme freshness of the sample, and though a cement in its fresh condition is unfit to use, it may be a perfectly good cement when aged, and in order that a cement should not be condemned unjustly it is advisable, in the event of a cement showing a tendency to blow on the first experiment, to lay some of it out in a very thin layer on a tray, so that it may be thoroughly cooled, and in the course of a few days another pat should be made and treated in a similar manner ; if this pat goes through the ordeal successfully and is perfectly sound, it may be fairly assumed that the cement only requires ageing to be a perfectly useful one; if, on the other hand, the second test proves unsatisfactory, it would not be advisable to use the cement. A cement may show indications of blowing while it is on the rack in the moist heat of the vessel ; if this happens it is needless to say that no corroborative test is required, the cement must be absolutely worthless.

The ordinary practice of carrying out this test is to make the pats in the morning, at, say, 10 or 11 o'clock, and to place them in the upper part of the vessel, and before leaving in the afternoon, say, at 5 o'clock, to put them in the water underneath, and to examine them for soundness the next morning, so that in 24 hours after the receipt of a sample its soundness may be known ; and the author feels sure that both users and manufacturers will agree with him in the importance and value of the test.

It is hardly possible to dismiss the subject of the soundness of cement without reverting to a test that was suggested some three years ago by M. Deval, and which was reported upon by M. de Chatelaine, and known as the "hot test." It consisted in gauging briquettes in the ordinary way, either neat or with sand, and when they were set, placing them in water which was kept at a temperature of 80° Cent. (*i.e.*, about 177° Fahr.), and it was maintained that, by so treating a briquette, the strength due to 28 days, as carried out in the ordinary way, was attained by this method in considerably less time, and thereby the constructive value of a cement could be more quickly ascertained. It was also maintained that this treatment of a cement determined whether it was a sound cement or not, for if the briquettes did not stand this excessive temperature, but cracked or became soft, then it was asserted that the cement was an unsound one.

When the author devised his apparatus for determining the soundness of a cement, which has already been described, he naturally had to make a great number of experiments before deciding on a temperature which it was advisable to adopt, and he then found that although some cements would bear being almost boiled, many cements that were in every respect good and sound cements would not stand the moist atmosphere and subsequent warm bath if the temperature was higher than that which he adopts, viz., 116° Fahr. for the bath ; he, therefore, when M. Deval's test was made public, made a long series of experiments to satisfy himself that he had made no false deduction in his previous experiments. The conclusion which he arrived at after these experiments with the "hot test" were the following :

1. That if a cement was really blowey, his own apparatus showed it equally with the hot test.
2. That the induration of a good cement was hastened as much, and sometimes more, by immersing the briquettes in water

maintained at the comparatively low temperature of 116° Fahr., as when immersed in a bath at the enormous temperature of 177° Fahr.

3. That no "fully" limed, as distinguished from "over" limed, cements would withstand the "hot test," but that all fully or over-clayed cements would stand it, and that consequently the test acted prejudiciously to what is accepted as a good cement, and gave preference to the over-clayed and quick-setting ones.
4. That nearly any cement that had been aged sufficiently would stand the hot test.

As the result of these experiments the author came to the conclusion that the hot test could hardly be considered a satisfactory test; and as the test has not made any great headway with either users or manufacturers, it seems that the conclusion he arrived at was fully justified.

In conclusion the author begs to submit the following notes on sampling and cement testing for the consideration of cement users. The specification is one which he has now adopted for several years, and finds that it in every way satisfies the requirements and ensures the delivery of a good cement.

Sampling.—When it is required to take a sample of cement for testing, it is desirable, in order to secure a fair average sample, to take a small quantity from several out of every 100 barrels or sacks, or the equivalent in bulk, and mix them all well together before taking the quanity required for testing, the samples being taken well from the center of the sacks, barrels, or bulk, and not from the surface, as that portion may have been accidentally damaged. When sampling from fresh or new cement, it is always advisable to cool it by laying it out in a thin layer for a few days, before putting the test in hand.

Gauging and Manipulation.—To obtain the best results, the minimum of water should in all cases be used when gauging cement. Small experimental pats should be made with a weighed quantity of cement and a measured quantity of water, in order to determine the exact amount of water required to properly gauge the particular sample under consideration. Having arrived at this knowledge, a sufficient quantity of cement for filling a nest of moulds should be weighed out, and the proper amount of water added thereto. It should

then be gauged with a trowel to the proper consistency, and filled into the moulds, being lightly rammed and gently shaken in order to remove all air bubbles. The briquettes should then be smoothed off and placed on one side. The whole operation from the time of adding the water to the cement to placing the briquettes on one side should not exceed five or six minutes. The briquettes should be removed from the moulds at the expiration of 24 hours from gauging, and placed in water, where they should remain until due for testing. It is customary to determine the tensile strength at the different dates by the average of five briquettes at each date.

Three pats should be made on pieces of glass or other non-porous substance, and their behaviour watched under the following conditions : Pat No. 1 may be left in the air, and No. 2 should be put in water as soon as it is set hard. Pat No. 3 should be treated in the apparatus for determining the soundness of cement.

Specification.

No. 1. *Fineness.*—To be such that the cement will pass through a sieve having 625 holes (25^2) to the square inch, and leave only 8% residue when sifted through a sieve having 2 500 holes (50^2) to the square inch.

No. 2. *Expansion or Contraction.*—That a pat made and submitted to moist heat and warm water at the temperatures and in the apparatus already described shall show no signs of expansion or contraction (blowing) in 24 hours.

No. 3. *Tensile Strength.*—Briquettes which have been gauged, treated and tested in the prescribed manner, to carry an average tensile strain, without fracture, of at least 250 lbs. per square inch at the expiration of three days from gauging ; and those tested at the expiration of seven days from gauging to show an increase of at least* per cent. over the

* If a 28 days' test is required, the average tensile strength should be at least 450 lbs. per square inch, and it must be noted that the increase in strength developed between the different dates is an indication of the growing strength of the cement, and admits of an approximation being formed of its ultimate strength ; but it is impossible to lay down any hard and fast rule as to what the increase between the different dates should be ; a slow setting cement will probably increase 50% between the three and seven days' test, and 25% between the seven and 28 days; whereas, a quick-setting cement may increase but very little. All cements should, however, show an appreciable increase in strength between the different dates; but as the increase in strength is not so great with quick-setting cements as with slow-setting ones, the tensile strength of a quick-setting cement should be greater at the shorter dates than a slow-setting one. All cements, more especially quick-setting ones, become slower setting and generally improve in tensile strength with age.

strength of those at three days, but to carry an average tensile strain of at least 350 lbs. per square inch.

The strain should be applied to the briquette at the rate of 400 lbs. per minute.

It is needless to say that it would be possible, without much difficulty, to continue the enumeration of the properties of cements and to discuss their peculiarities for a considerable length of time, but the author thinks he has already mentioned the most important, and is anxious not to encroach too far on the time of the Congress. He would, however, like to add that he fully appreciates the high honor which it has been his privilege to receive at the hands of the Board of Direction of the American Society of Civil Engineers and trusts that the somewhat discursive essay which he has presented may lead to such a discussion as will advance the knowledge of the manufacture and the use of Portland cement.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

635.

(Vol. XXX.—October, 1893.)

TOPOGRAPHIC SURVEYS.

By HERBERT G. OGDEN, Assistant U. S. Coast and Geodetic Survey.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

The geography of the surface of the earth, defining the boundaries of the land, the extent of the oceans and seas, and the discovery of the sources of rivers, has always excited an interest in the more intelligent of mankind. It is but natural that beings endowed with the attributes of man should seek for knowledge of their environments, and that, as the master-minds become more highly cultivated, this knowledge will be sought with greater intensity. Volumes have been written on the earlier theories of the earth's surface, and there are libraries of works on the explorations and discoveries that have been made. They evidence the progress of geography; but it is not within my province to discuss them, as they are but the preliminaries, the beginning, of which topographic surveys may be said to be the con-

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

summation. The student of topography, however, may profit by acquiring a knowledge of the development of geography. He will not proceed far in his researches before he will be impressed with the great loss of time and labor that has been incurred in rectifying the errors of explorers and geographers. It is true, that, in the earlier centuries, and we may say even to within a century past, those who sought new conquests in the field of geography did not have the means of locating their discoveries with the precision that can be attained by modern methods, and that inaccuracies and misconceptions necessarily arose from the imperfect methods. But these are the errors most readily detected, as they are based upon observations that can be assigned a probable value. The more numerous class of errors, unfortunately, were not so readily adjusted; based as they frequently were on unverified reports, or sometimes simply the imagination, they have necessitated many perilous voyages and years of arduous effort to disprove them.

These difficulties have been experienced in delineating continents, oceans, seas, and all the grand features of Nature; and in recent years, in locating many of the works of man, especially cities that history teaches were once the pride of nations and the homes of a civilization of which the barest traces can now scarcely be found.

We are led to inquire, does history repeat itself? Shall the progress of our day be buried under the accumulations of centuries? Is the great American Republic to be traced by fragments of history and the legends of the age? Who can say, and is there one so bold as to predict what will come? We cannot control a future age, but this does not relieve us of the responsibility to ourselves and to those who come after us in whatever age, that the record we make of that which we have shall be true.

In topographic surveying, we continue the work of the explorer and geographer; the same areas are defined, but with greater detail; expression is given to all the features of Nature and the works of man. The results present to us a picture of wondrous forms in which we study the transformation of hills and valleys, the growth of rivers, and the surface resources of mother earth, in advancing economic problems involved in the progress of civilization. It is but a step more, with the aid of the specialist, and we reveal the treasures hidden in the rocks of the ages.

Topographic maps are valuable in military operations, to the student of geology, and to the engineer in the projection of railroads, canals, wagon roads, and all routes of communication between specified localities. They have special value in the study and development of economic questions: in a region to be opened for settlement they indicate graphically the most favorable localities to the pioneer; and when, in addition, they represent the character of the soil, woodlands and other characteristics, they may be recognized as authority in the projection of plans of colonization and development that cannot be surpassed. They are also valuable in regions that have been long settled, in perfecting plans for improvements that are not usually required in new districts; as, the conservation and utilization of water power for industrial purposes, improvement of roadways, extension of cities and villages, and planning many of those new enterprises that spring up with the advance of civilization. They are a necessity in the study of geology, and are valuable to students of many of the natural sciences, and frequently are essential to the ascertainment of correct results, and their intelligible publication.

Topographic maps, to meet these purposes, may be of varying degrees of accuracy, and contain more or less detailed information depending largely upon the characteristics of the region they represent. It is not necessary that a map of a new region should delineate the features with the greatest refinement; while, on the other hand, a region that has been long settled must be represented with an accuracy and detail that admits of no substantial error, to permit the work being used to advantage in the character of the improvements that arise under such conditions.

The principal nations of Europe have completed surveys that will generally subserve these purposes, or are now engaged upon such work. In some instances, the political conditions that have prevailed in the Old World necessitated cadastral surveys that were made with great accuracy. As a rule, this cadastral work did not represent the relief of the topography; but when it was found desirable to add relief to the artificial features, the cadastral work proved to be a basis that would control and facilitate the representation. These European surveys are based upon a precise triangulation, and are usually made upon a scale not far from 1:25000, sometimes slightly larger, and sometimes slightly smaller; excepting in mountainous districts, where the

scale is generally reduced to about 1:50000. In some instances, where the surveys have been inaugurated on smaller scales than 1:25000, the scale has been increased, to permit a generous treatment of the details. The Ordnance Survey of Great Britain probably stands unrivaled in point of accuracy and thoroughness. The work had been carried over a greater portion of the Islands on a scale of 6 ins. to the mile, when it was determined that a scale of 25 ins. to the mile would be more suitable for the purposes the survey was intended to subserve, and the survey of nearly all the fertile districts of England and Scotland has now been completed on this large scale. In this work little or nothing was left to the imagination of the surveyor; all lineal measurements are matters of record; all elevations have been controlled by accurate level lines; and bench marks have been left at frequent intervals for future use. These large scale surveys are the groundwork of all map publications, the details delineated in the original surveys being generalized or eliminated in the reduction to smaller scales, preserving generally only those features that can be clearly represented. The contour interval is 50 ft., though, in some instances, 25 ft. has been used on the large scales. The contours are published on some of the maps on the 1-in. and 6-in. scales, but are rarely given on larger scales.

The British have also extended their work to the Indian Empire in the establishment of the great Trigonometrical Survey of India, and have provided for topographic maps of great variety and value. There is a general topographic survey being conducted on the scale of 1 in. to the mile, supplemented in many localities by cadastral surveys on scales of 4 ins., 16 ins. and 32 ins. to the mile. This cadastral work is very elaborate, and it is stated by Captain Wheeler, in his Report on Government Surveys, that in the survey of the town and island of Bombay, about 22 square miles, the fields and open country were surveyed on scale of 1 in. to 100 ft., with the native town and fort on scale of 1 in. to 40 ft., the relief being shown by 10-ft. contours, at an average cost of $7 040 per square mile.

In the United States no consistent attempt was made to conduct a topographical survey over the whole area until the organization of the Geological Survey, about 15 years ago. Nevertheless, a large amount of work has been done in delineating the natural features with sufficient accuracy to prepare geographic maps, and in some sections

topographic maps, of a high order. The first work of the General Government in the development of the Territories, and which has produced the largest amount of geographic information, was the organization of the Land Surveys, for the division of the public lands into township sites. These surveys supply very little topographic information; they traverse the streams, but rarely furnish any indication of the relief of the topography or the artificial details.

The second organization was the establishment of the Coast Survey early in this century, although the work did not assume definite proportions until 1832. Some 40 years later this survey was charged with the extension of its geodetic work across the continent, and with the determination of trigonometrical stations in those States that required their use in the prosecution of geologic and topographic surveys within their own borders. The topographic work resulting from the operations of this bureau is confined to a narrow strip along the coasts, rarely more than three or four miles in width. The surveys were made upon scales of 1:10000 and 1:20000; and having been designed for the double purpose of producing charts for the benefit of commerce, and maps for the public defence, embrace all the natural and artificial details that can be clearly represented. The earlier surveys were very defective in the delineation of details; but as the work progressed, improvements were introduced in methods that brought it up to a high standard. The survey of the Great Lakes under the Engineer Corps of the Army, established in 1841, furnished topographic maps of the shores of the Great Lakes similar to the later work upon the coasts. The original surveys by the Coast Survey are now nearly completed, and the original work upon the Great Lakes has been published for some years.

During the period in which these established organizations were prosecuting their labors on the coasts and Great Lakes, many explorations were conducted in the Territories, under the direction of the Corps of Engineers of the Army. The list of these exploring parties is a very long one, and as the results were mainly of a geographic character, although of great value at the time, they have little bearing on topographic surveys in general.

In the last thirty years there have also been a number of surveys organized for the development of special regions in the Territories that have resulted in the production of preliminary maps of specific topographic value. Among these were surveys by King, Powell, Hayden

and Captain Wheeler of the Army. These surveys represent little more than the mountain ranges, mountain passes, principal streams and valleys, but omit so many details of these features in the generalization that was resorted to, that the value of the work falls below the standard that should be attained in topographic surveys designed to subserve general purposes.

Incidental to work in the Interior, the operations of the Mississippi River Commission, established in 1876, and subsequently the Missouri River Commission, may be mentioned as furnishing a basis for further developments in the valleys of these great rivers. The courses of the streams have been triangulated, and a strip of topography obtained along either bank of the Mississippi River, representing the features in great detail. Similar work has been conducted on the Missouri River, but not with the same care in reference to details.

The Geological Survey, established in 1878, has extended its topographic work to nearly every section of the Union. In the Territories and Western States the omission of details of the natural features subjects the result, in a measure, to the criticism applied to the independent surveys that preceded it in the same regions. In the Eastern States, especially in those surveys conducted in co-operation with the State governments, there is a much fuller representation of both the natural and artificial details; the detail is more complete also in those States where the land has been subdivided by the Land Office; but under the general rules adopted by this bureau to omit all artificial features not clearly of a public character, and all wood outlines, there are many details of value to settled communities that are not represented.

This brief review of the topographic work in Europe and the United States leads us to consider what is yet to be accomplished in our own country, to supply us with general topographic maps that shall be of the greatest value. Heretofore our surveys in the Interior have been executed without a standard of comparison. The general scheme of the work has, perhaps, been prescribed to the surveyor, and in some instances the character of the details that he should represent or omit; but the selection of the details and the relative accuracy of their representation has generally been left to so large an extent with the individual on the ground, that we frequently find incongruities, and are greatly misled in interpreting the maps. To secure the production of uniform

work from many hands, it is essential that there should be a criterion to gauge the sufficiency of the surveys. A criterion has been formulated by a conference of topographers that assembled about a year ago at the Coast and Geodetic Survey Office in Washington by direction of the Superintendent of that Bureau, to define certain matters relating to topography falling under his own supervision. In the investigation of these subjects, the inquiries that were made included the work of European governments as well as that of our own Government. The material collected furnished data that suggested the propriety of formulating rules to govern the representation of the details of a region on a specified scale. While it is not the expectation that these rules will meet with universal approbation, it is believed they will indicate the requirements desirable to produce uniform work, and will lead to a consideration and discussion of the questions involved that will ultimately result in the adoption of a criterion that will be acceptable to all.

The problem presented some peculiar conditions arising from the general and specific purposes the surveys would be called upon to supply. There has been contemplated since the organization of the Coast Survey the probable extension of its topographic work, to such distance from the coast as would meet the general purposes for military defence. The execution of topographic work with this specific object in view, as well as the details requisite for purposes of navigation, necessitated the division of the detailed rules prescribed into two divisions: first, relating to the coast, and second, to the interior. The coast topography, embracing only a narrow strip along the shore, has heretofore been executed with a care that presented every detail that could possibly be required for the double purposes of navigation and public defence. As the work shall progress from the coasts, it is believed that much of this detail can be omitted without any serious detriment, and, in view of the decreased scale that can be used, may even prove to be advantageous. As the coast division refers to work to be executed for specific purposes, reference will be made to the rules for general topography of the Interior only.

All topographic work is to be based upon a computed triangulation. Experience has demonstrated this to be essential for preserving distances and directions over extended areas; small areas may be covered with graphic triangulation or filled in by direct measurements and be

properly checked by the computed triangulation surrounding them. All natural features will be delineated with greater or less accuracy depending upon their importance in the type, and a selection is made from the artificial features governed by their character and prominence; thus all roads that are open to the public, or that are used as highways, and all buildings that are on prominent locations will be indicated; while private farm roads, temporary wood trails, and, generally, fences will be omitted. The surveys should be made on a scale of 1:30000 with the exception of mountain types and flat areas with little natural detail, for which 1:40000 is recommended. The Conference was strongly of the opinion that 1:40000 scale was the smallest on which a reliable representation could be made under the rules that were formulated.

In considering types for the interior work, New England was selected as representing the greatest variety of natural and artificial detail. Rules for this type were prepared, and were taken as a basis for all other types, receiving such addition or modification as was found desirable. Passing over New England, a second type was found in the Allegheny Mountains, and other forms in the great Atlantic Coastal Plain, the Prairie Lands of the Central States, the Great Plains of the West, the Plateau Region of Kentucky and Tennessee, the Rocky Mountains and southern part of the Sierra Nevada, and finally the densely wooded Sierra Nevada of the North. These types of country have been designated by proper names, and rules formulated to govern the detailed representation of each. There is no feature, either natural or artificial, that can be fairly claimed to be common to all these types of country, and yet there are many features that are common to several of them. This distribution of the details that require representation renders it quite evident that no universal rules can be prescribed, and that a criterion to judge the sufficiency of any one type would probably be unsuitable to judge any other type. Additional complications will arise where two or more types are found in the same region; there may, therefore, occur a question where dividing lines are to be understood in representing two types on the same survey; but such questions seem unavoidable.

The instrumental methods to be employed on the surveys are also important factors. The Conference indicated its preference in methods for the different types; but methods must necessarily be modified by special conditions that may control at the time, and the ability and

training of the surveyor who may be engaged to perform the work in the field.

The natural features that it is desirable to represent on a general topographic survey may be classified as hills, valleys, plains, water-courses and woods ; and the artificial features as roads, water-ways, settlements and domiciles. These classes may be further subdivided in ways that will naturally suggest themselves. The rules to govern the selection and location of these features constitute the criterion. There is no rule that can be applied rigidly to all types ; but the principles involved in the rules are nevertheless of general application. The roads and water-courses usually form a framework on which to represent the other details of the survey. They may be treated under a general rule requiring a close approximation in their location, but permitting considerable latitude in less important localities. Generally, crossroads are to be shown in their true position, and no part of a public highway will be more than its width out of its true position. With less important roads, like those through forests, or by-roads, a larger error may be permitted—perhaps 50 or 100 meters in some instances. With the water-courses, the general system to be applied is great accuracy where the streams join, and increasing latitude as the importance of the streams decreases. In some instances, the width of the stream may be increased or diminished by small quantities, and for unimportant brooks passing through woodlands the general direction only is to be maintained, sinuosities of the brook being entirely ignored. A similar brook, however, passing through open land, requires that its course should be more closely delineated. Cities, large towns and compact villages are to be represented in a conventional manner, the streets to conform as nearly as practicable to the true plans. This preservation of the plan is the more important in straggling villages where a difference from the true plan is liable to mislead. The houses of a settlement are of so little value as a feature that they are entirely neglected ; but in the open country houses frequently become landmarks, and it is therefore provided that the most prominent domiciles and even barns, when isolated, should be represented with the accuracy required for other fixed points. In a group of domiciles, however, or when situated in localities where they do not attract great attention, their precise determination is of so much less importance that the greatest accuracy is not required ; the location of

a house, barn, or other structure in a forest is desirable, as buildings in such locations indicate definite measures of distance and identify localities ; if near a road, they are to be represented in connection with the road, and would be governed by the same rule of precision as the road. Water-ways constructed by man follow the general rule for the location of the roadways. They are of great importance in some regions, especially where agriculture is maintained by irrigation, where they have been constructed for the supply of water for large cities or intercommunication between definite points.

The relief of the topography bears directly upon all these details. In modern practice the hills and valleys are delineated by lines representing planes of elevation, called contours or curves of elevation, a uniform height being interposed between each plane. This system presents the most valuable data, but may frequently lack force of expression. The alternative system of hachures, or the representation of grades by lines of shading from vertical, or in some instances oblique light, gives a more striking expression on a small scale, but utterly fails to furnish the valuable data that may be derived from contours. There are many accidents of Nature, however, that would not receive attention in any system of contours it is feasible to extend over a large area; these the criterion provides for in a combination of the two systems. The general elevation and form to be shown by the contours, and the intermediate features of sufficient importance to require expression that the contours would not include, to be added in hachures. The necessity for this may be exemplified in the representation of any mountainous region, with a contour interval of 100 ft., or, as may in some cases occur, 150 ft., where the general slope, without any attention to minor details or breaks of the ground, would be shown by a gradual rise, practicable for the ordinary means of locomotion; but between these intervals there may occur precipices, ledges, gullies and other features that would make a passage-way most difficult if not impracticable. These features would be represented by hachures on the contours, and give an expression to the difficulties of the ground that will rarely be misunderstood.

For the New England States, and, in general, through rolling country, the contour interval of 20 ft. has been found to give great satisfaction, and rarely fails to represent the essential details of the natural features. This interval has, therefore, been recommended for

the New England type, and is taken as a unit, so far as practicable, for all other types.

The heights of the hills are to be given within 5 ft. of the true elevation, and, in general, the crests and floors of the valleys are to be represented quite as closely. The curves, showing the slope, can be sketched with greater freedom, but should not be in error more than the curve interval. Auxiliary curves may be used to form marked features falling between the prescribed contours; but when such features can be delineated by a slight deflection of the regular curve of not more than 5 or 6 ft., it is desirable to represent them by this means, rather than by the introduction of auxiliaries. In mountainous regions, the rules give still greater latitude, the representation being governed, as before, by the curve interval, but the interval being five or six times as great. It is not contemplated, however, that these great intervals shall be carried over the foothills and rolling country at the base of mountain ranges, as they would fail to give that minute description of the smaller features that is desirable.

It is evident that with the relief represented in this degree of detail, the grades of the natural water-courses and the elevations of small lakes and ponds would be fairly well determined; the natural basins would also be delineated with an accuracy that would permit a close computation of their capacity, and it would only be necessary to gauge the streams to determine very closely the water-power available for industrial purposes and for the supply of cities and towns.

The wood outlines were conceived to be a feature that required representation, though not with such accuracy as fixed features; the outlines are subject to change, especially in newly settled regions; but in the older districts the liability to change is not nearly so great, and in general the wood areas represent in these districts the principal localities that are not so available for agricultural purposes. Again, in the western country, where large tracts are almost destitute of trees, the representation of the growth that is found is of great importance, perhaps as indicating the presence of water, or as forming a prominent landmark. Instances of this kind would naturally be represented quite closely in their true position and extent; but in regions where wooded growth is common, an outline true within 50 or 100 m. would be considered sufficiently precise.

Topographic maps, made on the principles laid down by the Conference, would probably subserve all general purposes. The practicability of completing such a survey within a reasonable period, and at a reasonable cost, will, doubtless, be raised as an objection to any such project. The time required would very largely depend upon the sum of money that would be available for its prosecution. With an unlimited supply it might be possible to train a sufficient number of surveyors to complete the whole area of the United States within a generation; but it is not probable that such haste will ever be necessary. Maps of this character, however, would be found so valuable in economic questions that all work executed for general topographic purposes should be gauged by a standard at least as rigorous.

It is exceedingly difficult to determine the cost of work that is the continuation of surveys that have been in operation for many years. There is always a tendency to improve—to take in some new feature or omit some feature that has heretofore been represented—and these changes may increase or diminish the cost to such a degree that the average results for long periods become totally unreliable. Moreover, the same character of work executed by different men will present different rates according to the capacity and experience of the individuals. Captain Wheeler, in his admirable report on Government Surveys, ascertained the approximate cost per square mile of the principal surveys of the world. He states his experience in endeavoring to secure data of this kind, and expresses the opinion that his results must be considered merely as estimates, though probably the best that can be prepared. Some of these estimates are very interesting.

The average cost of the Ordnance Survey of Great Britain up to 1881 is estimated at $186 per square mile; but it was thought that the surveys on the scale of 1:2500 when completed would exceed these figures, and probably reach about $244 per square mile. In the German Empire, on a scale of 1:25000, the work has cost about $79 per square mile; and in Austria work on the same scale of 1:25000, including the office operations, has been estimated to cost about $400 per square mile. The cost of revising work in Holland, on the scale of 1:25000, is put down at $16.50. In India, the work on a scale of 1 in. to the mile, taking the average for 10 years, cost $11 per square mile; the work on 2 ins. to the mile was $26, and on 6 ins. to the mile, $400 per square mile, without the cost of printed maps. The survey of Bombay, on

very large scales, as before noted, cost $7 040 per square mile. There is probably more confusion in these figures than appears on the surface, as questions relating to services are not clearly stated, such as the pay of officers in the regular employ of the different governments; services rendered by detachments from the military forces; incidental office expenses, and items of that character. These figures indicate, however, that the best experience of European governments requires an expenditure considerably larger than it has ever been thought prudent to expend upon any of the interior work of our own country. Probably the greatest cost per square mile of topographic work in the United States, aside from the surveys on the coasts and Great Lakes, and other specific surveys of a like character, has been made by the U. S. Geological Survey in the work undertaken by that Bureau in connection with the State governments; and as the accounts have been kept separate from other expenses, they differ from the usual statements relating to cost, and furnish reliable data for work of this character. The average cost in Massachusetts was $13 per square mile; in Connecticut, $9.80; and in Rhode Island, $9. These surveys were made on a scale of 1:30000, which presents very generally the same facilities for representation as the general European scale of 1:25000. Other topographic surveys by the Geological Survey of the same class of work, especially in the west, owing to the type of country and facilities afforded by the marks of the Land Survey, cost very much less than these figures and the estimate for the average cost of work throughout the whole of the United States, without Alaska, does not exceed $4 or $5 per square mile.

The topographic work of the Mississippi River Commission is more closely allied with the detailed surveys recommended for the Interior. The cost of this work per square mile is therefore a nearer approximation of what would be the probable cost of detailed topography. The report of the Commission specifies an area of 819 square miles surveyed, 182 square miles of which, however, was hydrographic work of the Mississippi River, at an average cost of $48 per square mile. Also that 1 135 square miles previously surveyed cost $57 per square mile. It is not clear whether the latter item includes any hydrographic work or not. The topography of the Coast Survey and the Survey of the Great Lakes was made on so large a scale and involved intricate details of shore line and other coast features not found in the Interior, that so

greatly enhance the cost that they cannot be directly compared with any other surveys in this country. General Comstock reported that the average cost per square mile on the survey of the Great Lakes was $151. This probably includes office expenses and many incidentals that would not appear in ordinary field accounts.

Several attempts have been made to ascertain the cost of topographic work on the Coast Survey, but it is so frequently involved with expenditures for triangulation and hydrography that it is exceedingly difficult to compute an average for any lengthy period of time. A late Superintendent pronounced the survey of Mount Desert Island to have been among the most costly; yet the expense for the topography he stated was only $97 per square mile. There are, besides, many instances that could be cited where the cost ranged from $60 to $75 per square mile for work on the New England coast, and $10 to $15 on the southern coasts. Referring, then, to the operations under European governments and under our own, exclusive of work on the coasts and Great Lakes, it seems quite probable that the survey of the whole United States could be conducted on scales of 1:30000 and 1:40000, giving the detail that has been outlined, and with the accuracy that is called for by the rules recommended by the Conference, at an average cost of from $35 to $45 per square mile for the most difficult and detailed parts of the work.

The topographic work that has heretofore been conducted by Government Bureaus, either abroad or at home, has shown a tendency to improve as the work progressed. The persons in charge of surveys of this character are naturally ambitious to increase the value of the work under their direction and introduce new features suggested by their own experience and the experience of those working under them, and by the criticisms of those who make use of the publications. Changes arising from these sources frequently involve the mode of representation, the scale of publication, and even the scale on which the original surveys are to be made, and sometimes have necessitated radical changes in the methods of making the surveys. In Europe cadastral surveys have been improved upon to produce topographic surveys; small scale work has been abandoned for larger scale work, and the general tendency has been to present all detail with greater accuracy than was originally contemplated. New instruments have been devised in the hope of accomplishing this without unduly increasing the

cost. Improvements of a like character have been made in the United States; the early explorations and geographic surveys of the western territories have been superseded by an organized topographic survey, which, in turn, has developed its methods, to delineate more closely the features represented, even enlarging the scale for some types that experience has demonstrated required a larger scale to bring out greater detail of the features.

The improvements that have been made in the topographic work of the Coast Survey probably exhibit the development of such work more clearly than will be found in any bureau of the Government service. The history of the development of our methods is interesting and has not heretofore been published.

At the practical beginning of our work in 1832, we had as a guide only the experience of a few men who had practiced topographic surveying in the Old World, and reliance seems to have been placed without question on the sufficiency of the methods they introduced. Their methods were continued in the actual work of the survey for some 10 or 12 years when suggestions from a young officer produced the first change, which has led by gradual steps to the perfection that has now been attained. The plane-table has always been used on the Coast Survey in the topographic surveys, but the knowledge of the best methods of utilizing the instrument was so imperfect that the surveys did not acquire a high standard. The instrument was used principally as a substitute for the surveyor's compass ; the advantages to be derived from using it as a triangulation instrument to interpolate graphically subsidiary stations from a computed triangulation having been at first almost entirely ignored. The delineation of the topographic features resulting from these imperfect methods was necessarily defective ; the hills were represented by hachures sketched with the greatest freedom ; the shore line presented only the most salient points with any great degree of accuracy, the bights and coves being sketched with almost the freedom of the hill work. The artificial features were proportionally defective and sometimes aggravated by a system of representing the cleared lands suitable for agriculture, by a confusion that drew imaginary fences over the whole area. The first great improvement was in the method of using the plane-table. The theory of the instrument was, of course, well understood ; but the practice had evidently been greatly neglected. The three-point prob-

lem, as it is called, or the graphic method of finding a position from three other determined positions, was brought forward and utilized in securing more accurate determinations of the sketching stations. The second step was a more precise delineation of the form of the hills, or the relief of the topography, which was soon followed by an effort to give absolute elevations as well as forms. Some of the young topographers, in following the old system of representing the hills by hachures, had found it convenient to first shape the hills with parallel lines, or contours as they were then called, though not representing planes of elevation as our curves do in the present day. By means of these contours they easily saved the time consumed in the field in drawing the hachures, relegating the labor of hachure-drawing to the office after the completion of the field work. It was only a step from contours to curves of equal elevation; these were soon proposed by one of the young topographers who had been using contours to shape the hills, but, being an innovation, seems to have met with decided opposition. Permission was granted, however, for him to try his scheme of representing the forms of hills with lines showing approximately every 20 ft. of elevation, the hachuring that might be subsequently required in publication to be executed by a draughtsman. At first the curves of elevation depended entirely upon the eye of the topographer for their distance above water level, and were looked upon as simply a new method of representing the forms of the hills, or a substitute for hachures. Up to this time the elevation of the hills had not been ascertained; the simple fact that here there was high land, with the approximate form and area, was all that was recorded. But the curves of elevation were found to be so much more convenient, and represented so much more clearly the relief, that the attempt was soon made to determine the elevation by actual measurement, and for this purpose a small, vertical arc was added to the alidade of the plane-table. This addition to the alidade was so effective and reliable that it is still in common use.

The system of representing the relief of the topography by curves of equal elevation was discussed in Europe in the early part of the century, the discussion running for a period of 20 years, and with such earnestness that it became known among topographers as the "Twenty Years' War of Hachures and Contours." The victory seemed to be for a time with the advocates of hachures; but the evident advantages

of the contour system finally prevailed, and between 1840 and 1850 it was receiving very general recognition. It is merely a coincidence that about this time the same system was developed by American topographers on the work of the Coast Survey, and, as the writer has every reason to believe, without any knowledge of the discussion that had long continued in Europe.

The method of topographic work on the Coast Survey had now reached a development that required only experience on the part of the topographers to produce results of ample refinement for the purposes of the survey; but subsequent demands on the Bureau for special surveys, with the requirement, in many cases, that the curves of elevation should have the same relative accuracy as the artificial features represented upon the maps, necessitated a modification in the details of the system of work, and the introduction of auxiliary instruments. These were supplied on large scale surveys, where this increased accuracy in the relief was demanded, by running out each contour with a surveyor's level from established bench marks, and determining each position of the leveling rod on the plane-table sheets at the same time. Numerous surveys of small extent have been made in this manner, and recently there has been completed a topographic survey of the District of Columbia, embracing about 60 square miles on a scale of 400 ft. to the inch, that is doubtless as accurate and thorough a piece of work as has ever been executed over such an extended area on so large a scale. It has been severely tested by the engineers of the District Government and by civil engineers interested in the subdivision of suburban property, and has uniformly been accepted as work of the highest order. It represents every detail upon the surface, with contours for every 5 ft. of elevation, reaching a maximum height of over 400 ft. These contours develop the cuts and fills in the roads, gullies and similar features with great accuracy.

In considering questions of topographic surveying, we must not confound the general survey with surveys for special or specific purposes. The general survey may simplify the projection of special work, but can in no sense supplant it except for general purposes. To determine the exact line for a railroad or canal, or to furnish the data for computing the labor of constructing a dam, and for other similar purposes, the special survey will always be required; but it is true, also, that in a general survey the local conditions and the most prob-

able uses of the work may properly receive consideration. In a region with an abundant water supply we would naturally omit giving attention to problems of irrigation; while, on the other hand, in regions where a natural water supply is deficient, we would pay the closest attention to the delineation that would exemplify the resources of the region for irrigation. Again, great mountain masses would be greatly generalized in a region that is difficult or perhaps impossible of access, while the passes, and the approaches to the defiles, would receive such careful attention that their availability as routes of communication would be thoroughly portrayed.

The most advisable form for the publication of a topographic survey should depend upon the character of the survey and the special purposes the publication is intended to subserve. The first conditions, therefore, to guarantee publications of the greatest value must be the faithfulness of the field work and its original representation on a sufficiently large scale to show the details clearly. A base map of this character can be reduced to smaller scales, and still preserve the essential details for general purposes; can be copied on the original scale; or, if need be, enlarged for special purposes, and can be generalized for geographic purposes. In each case the information conveyed would be reliable, measured by the scale and modified only by the faithfulness of the surveys. An argument is not needed to present the usefulness of trustworthy maps. Few men engaged in the active enterprises of the day have not experienced the demand for them, and we may safely predict that, as our people advance in prosperity, the demand will come from the masses. Already in some of the older States the demand has taken form, and other States are knocking at the door. Such surveys are essentially national; they benefit all communities in enlarging the horizon and bringing before the people knowledge they can attain in no other way; although it is true also that some particular communities may gain the greater benefit for the time being. It may be right, therefore, that the expense for such work should be borne jointly by the National and State governments; but to secure uniformity it is essential that the surveys should be supervised and conducted by men trained for the purpose; that the experience gained by the first who undertake such work shall be transmitted to their successors by contact and practice. One bureau of the National service is now authorized to co-operate with the States and furnish geograph-

ical positions; another is engaged upon a topographic survey for geologic purposes, and at the request of some of the States has jointly with them improved its topographic work within their boundaries to the extent the joint appropriations would permit. But the results thus far produced by the surveys that have been made in the Interior do not meet the requirements of a general topographic survey: either the work has not been supported by sufficient means; the attempt has been made to accomplish too much in too brief a time, or the specific purpose held in view has restricted the operations to an extent that prohibited the treatment required by a general survey. It is quite possible some of the older States will soon meet with economic problems that will force them to obtain maps with a more generous supply of information, and we may feel assured that, when the advantages of such maps have once been demonstrated to our people by actual experience, they will fully appreciate them and incite a demand for their extension. The day, the writer believes, is not far distant, when we must seriously consider the best means of conducting topographic surveys in all the States that will insure us the most valuable results for the expenditure. There can be no doubt that all the maps should be directly comparable; that while some might have a superior excellence, there should be none that would fall below a certain standard. The reputation for reliability attained by the surveys on the coasts, purely the result of providing by law the objects the work was to subserve and the methods that should be employed, leads the writer to believe that, to secure the desired harmony of results in all the States, the surveys should be conducted under rigorous rules, prescribed in a National law.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

636.

(Vol. XXX.—October, 1893.)

RECENT EXPERIENCE ON THE U. S. COAST AND GEODETIC SURVEY IN THE USE OF LONG STEEL TAPES FOR MEASURING BASE LINES.

By R. S. WOODWARD, C. E.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

Introductory and Explanatory Remarks.—Few if any appliances in the art of measurement on the surface of the earth are of higher antiquity or greater utility than the metallic chain, now commonly called, among English-speaking peoples, the surveyor's or Gunter's chain. No appliance is more widely or better known than it, and none meets with more general appreciation. The compass, a much later invention, to which the obscure property of magnetism adds an element of mystery, is undoubtedly an instrument of more profound interest to the curious and thoughtful; but the greater antiquity, the incomparable simplicity and the more general usefulness of the chain entitle it to the place of first historical importance in any enumeration of such ap-

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

pliances. To the surveyor whose measurements determine the metes and bounds of properties, to the property owner who must depend on such measures, and to the jurist who decides between conflicting interests, the chain has long been and still is the most common unit of definition and limitation.

Simple and effective, however, as the chain is in its way, it fails to meet many of the demands of the more refined work of the present day. With the growth and consequent complication of property interests, there arose a need for increased precision in the definition of distances. And as the engineer has succeeded the surveyor in such higher grades of work, so have other devices taken the place of chains. Among these devices the most important as regards general utility are metallic tapes and wires. The adaptability of tapes, especially, to a wide range of measurements is now well known. Indeed, their use has become alike indispensable to the engineer, the architect, and the artisan. The superiority of the tape over the chain, when precision is in question, is manifest. The defects due to the numerous links and joints of the chain are overcome by the continuity of the tape; while the advantage of pliability secured by the joints of the former is met by the elastic flexibility of the latter. Hence the tape must be considered the natural successor of the chain; and in any scale of relative usefulness and precision the tape, as ordinarily used, must rank next above the chain.*

In many, if not most, of the applications of linear measures it suffices to ignore the effect of heat in changing the lengths of our standards. When we take this effect into account, however, the order of precision attainable is greatly increased. This fact has long been adequately recognized with reference to bars used as standards of lengths; but that the same heat-effect constitutes the principal factor of length change in metallic tapes and wires seems to have been rather tardily appreciated; for, although the art of wire-drawing dates from the latter part of the fifteenth century, the art of using a wire or tape in measures of precision is hardly more than two decades old. Within this latter period many persons have independently entertained the idea of using long tapes and wires for the more refined work of geography and geodesy. The credit of calling distinctive attention to the availability of tapes and wires in such work appears to belong to Prof. Edv.

* It appears that metallic tapes were invented and introduced by J. C. Chesterman, to whom Letters Patent were granted by the English Government in 1842. He had previously patented measuring tapes made of cloth or other fabrics in 1829.

Jäderin, of Stockholm. His researches began in 1878, and his memoir,* published in 1885, in which he gives a full account of the theory and results of his measures, has been an important guide to subsequent investigations. The essential fact developed by Jäderin, and the fact on which attention should be concentrated in considering this subject, is that the temperature of the tape or wire is the most important factor in determining its length. When this fact is recognized, it becomes evident that if means can be devised for getting that temperature, the tape combines the advantages of great length and simplicity of manipulation of the chain, along with the precision of the shorter or laboratory standards.

It is the object of this paper to give a brief account of the recent experience of the writer in the use of long tapes for measuring base lines in the primary triangulation of the U. S. Coast and Geodetic Survey; to explain the processes developed in this work, and to give such results as will enable the reader to judge of the merits of the method for geodetic work.

The investigation of the practicability of using long metallic tapes for measuring lines of precision was taken up by the Coast and Geodetic Survey in the autumn of 1890. This work was assigned to the writer, who conducted the preliminary experiments and devised the apparatus described herein. In all of this work very efficient assistance was rendered by Mr. J. S. Siebert, who made most of the working drawings for the construction of the apparatus and participated equally in the large amount of observation and computation entailed.

Steel tapes 100^m long were used during the summer and autumn of 1891 to measure the Holton Base, Ind., a line in the Transcontinental chain of primary triangulation. The five sections of this base (of $5\,500^m$) were measured from 6 to 30 times each, under various circumstances as to temperature, time of day, etc. In the following year, viz., October, 1892, another base (of $3\,900^m$) at St. Albans, West Va., in the same chain of triangulation was measured. The results for this latter base are given below.

Mercurial thermometers were used in all these measures to determine the temperature of the tape. An attempt was made in 1891 to use a bronze tape in connection with the steel one for temperature in-

* Geodätische Längenmessung mit Stahlbändern und Metalldrähten. Von Edv. Jäderin. 8vo, 57 pp., 2 plates. Stockholm, 1885.

dications; but the bronze tape was found to have too low a rate of expansion for successful competition with the mercurial thermometers. The feasibility of stretching two tapes simultaneously was demonstrated, however, and if a suitable metal for use with steel can be found it may prove advantageous to discard the mercurial thermometers.

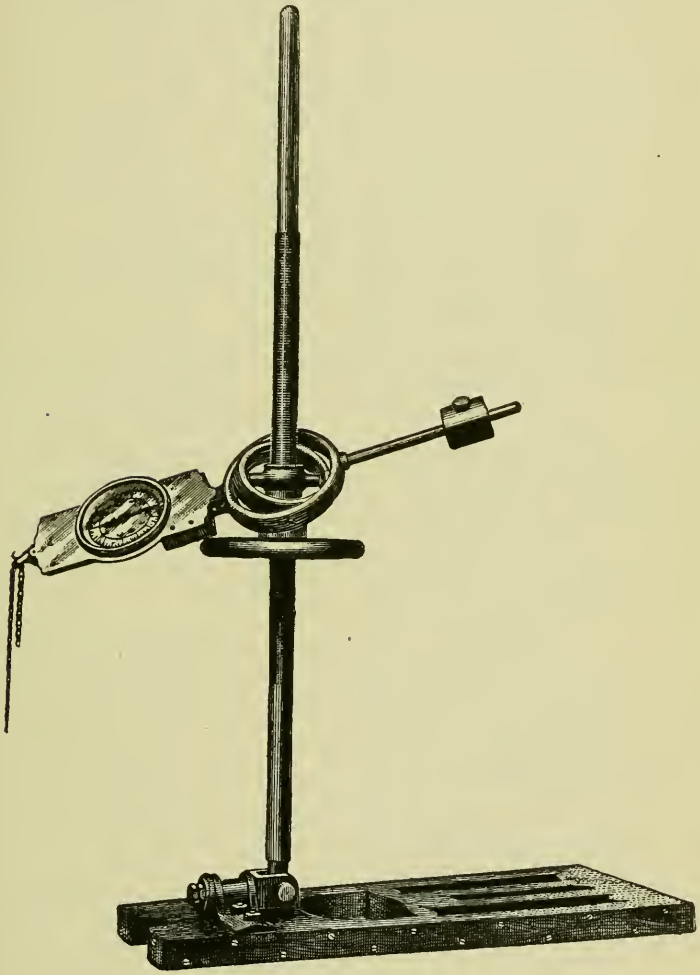
During the winter of 1890 and 1891 a great number of experiments were made with a view to determine the best method of stretching and aligning a tape, and especially with a view to discover the most effective way of getting the tape's temperature. For this purpose a 100^m comparator was set up in the Botanical Garden at Washington, D. C. This comparator consisted simply of a series of equi-distant stakes, 10^m apart, bearing support nails for the tape, and a post at either end on which were attached scales for defining the tape's position at any time. These preliminary experiments served to fix attention on most of the points essential to the use of tapes in precise measures.

Description of Apparatus and Methods of Use.

The 100^m *Tapes.*—These tapes are of steel. They are 101.01^m long, and are 6.34^{mm} x 0.47^{mm} in cross-section. They weigh 22.3 gr. per meter of length. They are subdivided into 20^m spaces by graduations ruled on the surface of the tape itself. Being 101.01^m long, the end graduations fall about 0.5^m from the tape ends, which terminate in loops formed by annealing and riveting the tape back on itself. The surface of the tape, where it is not polished to receive the graduations, is of a dull black color. When not in use the tapes are rolled up on reels, and they may be thus transported with ease and safety.

The Tape-Stretchers.—The nature of the tape-stretchers used to give tension and alignment to the tape may be best understood by a glance at the accompanying cut. They consist of a lever hinged by a universal joint to a platform on which the operator stands. This lever is made of a piece of steel tubing terminating in a hickory handle. Along the upper two-thirds of the tube is cut a screw-thread on which a wheel-nut plays freely. This nut gives a vertical motion to a gimbal-jointed support to which a spring balance is attached. The balance is connected with the tape by means of a short piece of sash chain. It is evident from the figure that the stretcher has all the motions necessary to enable the operator to align the tape and to give it the proper tension.

The balance, which is simply a commercial article adapted to the purpose, reads to ounces, and may be easily held to the nearest ounce by the operator.



An important item for the safety of the tape used is a breaking link, or a link which will part under a tension of about 14 kg. or 30 lbs. A link of this sort is provided at each end of the tape, so that it cannot be overstrained by accident.

It will be noticed that the balance carries two hooks. The extra hook, which is attached to the balance frame, was intended to carry the additional tape in case it proved advantageous to use tapes of different metals simultaneously. This plan of stretching two tapes worked very satisfactorily, the tension of the steel tape being given by one balance, and that of the bronze by the other; but for the reason stated above, this plan was followed only in making a few of the tape comparisons mentioned below. The extra hook, it may be stated, is provided with a longitudinal screw motion, so that by means of this and the links of the connecting chains the tapes may be always brought to the proper tension whatever their temperature.

When a single tape is used, the rear end is attached to the extra hook of the rear end balance, while the desired tension is given by the balance at the front end of the tape.

An objection to this form of stretcher lies in its weight, which is about 45 lbs. Contemplated improvements will lighten it by several pounds and bring the balance to a position concentric with the lever shaft. Experience has shown, however, that an able-bodied operator can handle the stretcher successfully when measuring at the rate of 2 km. per hour. During the measurement of St. Albans Base, in 1892, 8 km. were measured on one occasion between 7.30 P.M. and midnight without excessive fatigue to the operators.

The Thermometers.—The thermometers used are of the Centigrade type and are graduated on their stems to half degrees. They were made by Green, of New York. They were originally mounted on brass scales. The preliminary experiments showed that the mass of these scales caused the thermometers to lag very considerably with respect to the tape temperature. Hence the scales were removed, and light wire loops were attached to the upper ends of the stems, so that the thermometers could be suspended by a cord and thus isolated from adjacent masses or whirled in the air when necessary. Experiments also showed these thermometers to be slightly more sensitive when the bulbs were covered with sheaths of blackened aluminum foil, though this increase, if real, is probably unimportant. During the measurement of St. Albans Base, in 1892, the aluminum sheaths were replaced by steel sheaths, made by coiling and annealing very thin steel tape into lengths just long enough to slip over the bulbs and clasp the thermometer stems. These are undoubtedly superior to the aluminum

sheaths in simulating the surface of the tape, but they are of somewhat greater mass than the aluminum sheaths and cause more lag of the thermometers with respect to the tape. In any case it seems desirable to blacken the bulbs of thermometers used with steel tapes presenting a blackened surface.

A great number of experiments, made in the Botanical Garden during the winter of 1890-91, showed that with the thermometers arranged as just described (*i. e.*, with aluminum sheaths) they followed the tape temperature without serious lagging when observations were made under a clear sky at night. Subsequent experience in determining the tape lengths on the 100^m comparator of Holton Base showed a very satisfactory accordance of the tape and thermometer temperatures, whether observations were made during the day or night. This comparator was covered, however, except on its north side, by a shed which screened the tape from the direct rays of the sun. The results of these determinations of length are given in detail below, in Tables Nos. 1 and 2.

A striking fact brought out by repeated measures of the sections of Holton Base on cloudy days is that at such times the tape temperature was persistently higher than that indicated by the thermometers. The cause of this peculiarity is not yet fully made out, though investigations are under way to discover it. The amount of the discrepancy in our experience has risen at most to 0.9° C., which is equivalent to $\frac{1}{110000}$ part of the tape length or distance measured.

Sometimes three of the thermometers were used with the tape, and at other times two. When three were used, they were placed one at the middle of the tape's length and one at one-sixth that length from either end. When two were used they were generally placed at one-fourth the tape's length from its ends respectively. For the most precise work, as in comparisons for determining the tape lengths, the three thermometers were each read twice, once just before and once just after the observations on the tape. The mean of the six thermometer readings thus made gave the tape's temperature with a probable error of about $\pm$ 0.2° C., equivalent to $\frac{1}{500000}$ part of the tape length, when observations were made at night or under the comparator shed.

Method of Supporting and Marking Position of Tape.—When in use for measuring a line the tape is supported at equal intervals of 10 or 20 m. throughout its length. The supports found most convenient, and

amply sufficient, are steel wire nails driven into stakes set at the proper intervals along the line. These nails are ranged into a straight line for any tape length by means of a small theodolite whose telescope axis may be brought to the right height at one end of that tape length.

The ends of the tape are supported by the tape stretchers, by means of which the tape is aligned over the marking stakes. The latter are stakes of scantling cut off or driven down to the proper height and capped with a small table on which a plate of zinc is held in place by light nails. These tables and plates are rectangular, 5^{cm} x 20^{cm}, say, and the longer sides are placed parallel to the line. On the plates the position of the forward-end graduation of the tape is marked as the measure progresses by means of a sharp brad-awl held against the edge of a small try-square, which is aligned against the longer edge of the marking plate. The rear-end graduation of the tape is in turn brought into coincidence with the successive marks on the zinc plates.*

Operation of Measurement.—The marking and support stakes for the measurement of a line are most advantageously ranged out and adjusted beforehand. This is especially essential for work of the highest precision carried on at night. Assuming this to have been done, and that night work is contemplated, the operation of measurement, if carried on rapidly, will require 12 men, to wit: two observers of the tape, one at the front and one at the rear end; two observers of thermometers, who also help to carry the tape forward; one recorder; two operators of the stretchers; five men to handle lamps and to carry the tape forward.

The co-operation of the observers and other operatives is secured by means of a code of signals made with whistles held by the observers. Thus, when the observer at the rear end of the tape is ready to have the tension applied, he gives one blast of his whistle. The stretcher at the front end immediately brings his balance to the proper reading, when the observer at that end announces the fact with one blast of his whistle. With the tape under tension the rear-end observer adjusts the rear-end graduation of the tape to coincidence with the defining mark on the zinc plate at that end. When this is accomplished he gives two blasts from his whistle. On hearing these, the recorder,

* The use of such plates was introduced in 1885 by Mr. O. B. Wheeler, Assistant Engineer Missouri River Commission.

who stands at the middle of the tape length, immediately lifts the tape a few centimeters from the support nail near him and lets it fall back. The effect of this is to make the tape straight and relieve it from friction on the support nails. When the vibration of the tape ceases, the rear observer gives one blast, which signifies that the position of the front end of the tape may be marked on the zinc plate. The mark is made by the front-end observer, who, on completing the operation, gives one blast from his whistle. Then the thermometer readings, which are observed while the mark is made, are called out to and repeated by the recorder. Finally, the rear observer sounds two blasts from his whistle and the tape and tape stretchers are carried forward to a new position.

The speed attained in this process has been about 2 km. per hour, though a single kilometer has been measured in 20 minutes.

By reason of the expansion and contraction of the tape, it is frequently desirable to set the tape back or forward by a round number of centimeters. This may be done on the zinc plates by means of a suitable pocket scale. Such displacements are recorded as "set-backs" and "set-ups." Their amount is engraved on the plates, which may be appealed to if any doubt arise concerning the record. When "set-backs" or "set-ups" are made, and when more than one measure is recorded on the plates, it is advantageous to number, orient, and date them for filing as part of the records.

Determination of Equations of Tapes.

Data for Tape Lengths.—For the purpose of getting accurate lengths of the tapes and also testing their efficiency in the actual work of base measuring, a new form of base apparatus, known as the iced-bar apparatus, was devised by the writer. The essential feature of this apparatus, which is described in the *American Journal of Science* for January, 1893, is that the measuring bar is kept at a constant temperature by means of a packing of melting ice, thus avoiding in its applications the use of thermometers entirely. It is a line measure, micrometer microscope apparatus, and it appears to give a precision of measurement surpassing anything hitherto attained in base-line work.

The tape lengths were derived from numerous observations on a 100^{m} comparator whose length was determined by repeated measures with the iced-bar apparatus. The tapes were aligned and supported

on the comparator in precisely the same manner as when used in base measurement.

The details of all the work done up to date with the iced bar and with the tapes are now nearly ready for publication in the report of the United States Coast and Geodetic Survey. But since the character of the work done in deriving the equations of the tapes can be gauged adequately only by the results obtained, it is proper to anticipate that report to the extent of giving the essential details of the tape comparisons. The following tables give those details for the 100^m tapes Nos. 85 and 88 respectively. The first column gives the date of the comparison; the second, the time of day; and the third, the thermometric temperature of the tape in Centigrade degrees, corrected to the hydrogen scale. The fourth column gives the observed length of the tape in terms of the iced bar, the excess over 20 times that bar's length having been measured by means of micrometer microscopes and the millimeter scales of the cut-off cylinders accompanying the iced-bar apparatus. The standard tension of the tape is 11.595 kg., or 25 lbs. 9 ozs.; *i. e.*, it is the actual tension given to the tape when the balances used read 25 lbs. even. The last column gives the residual, or the difference between the length of the tape computed from its equation given below and the observed length.

TABLE No. 1.

RESULTS OF OBSERVATIONS FOR LENGTH AND RATE OF EXPANSION OF 100-METER STEEL TAPE No. 85.

Date.	Time of Day.	Temperature of Tape.	Observed Length of Tape.	Residual (computed minus observed value).
1891.				
August 6th.....	11.29 A. M.	29.02°C.	$20B_{17} + 35.51^{mm}$	$+ 0.12^{mm}$
	1.02 P. M.	30.56	37.69	— .34
	2.06	30.07	36.15	+ .63
	3.05	29.88	36.85	— .28
	4.02	28.92	35.14	+ .38
	5.00	28.01	34.58	— .06
	9.02	19.58	24.66	+ .63
	9.12	18.44	23.80	+ .25
August 7th.....	7.02 A. M.	20.96	26.84	— .03
	8.02	24.53	30.84	— .13
	9.02	26.73	32.97	+ .15
	10.02	27.96	34.57	— .11
	11.01	29.26	35.95	— .06
	11.48	30.11	37.54	— .70
	2.04 P. M.	30.21	37.28	— .35
	7.16	31.35	38.47	— .29
	7.32	21.51	27.04	+ .37
	8.38	20.15	25.31	+ .60

TABLE No. 1—*Continued.*

Date.	Time of Day.	Temperature of Tape.	Observed Length of Tape.	Residual (computed minus observed value).
August 8th.....	7.28 A. M.	24 66° C.	$20B_{17} + 30.43^{mm}$	$+ .43^{mm}$
	8.34	27.06	33.89	— .41
	9.35	29.14	35.89	— .22
	10.32	30.34	36.90	— .17
	11.31	31.39	38.39	— .17
	12.28 P. M.	32.12	39.23	— .21
	1.33	31.54	38.72	— .33
	2.34	30.32	36.77	+ .28
	7.41	21.48	27.10	+ .27
	8.34	20.93	26.56	+ .21
	9.34	20.38	26.20	— .03
September 5th.....	2.40 P. M.	14.88	20.20	— .05
	3.24	14.70	19.77	+ .18
	5.04	14.26	19.37	+ .10
	7.35	13.88	19.26	— .21
	9.08	13.98	19.12	+ .04
October 3d.....	10.40 A. M.	26.73	33.25	— .13
	11.48	27.24	33.54	+ .14
	1.41 P. M.	27.06	33.38	+ .10
October 4th.....	4.54	13.85	19.41	— .39
	6.50	12.67	17.93	— .20
	8.08	12.32	17.18	+ .17
	8.58	11.70	16.85	— .18
	10.20	11.38	16.18	+ .14
October 5th.....	5.00 A. M	7.20	11.66	+ .07
	6 04	6.66	11.19	— .04
	6.54	7.97	12.64	— .05
	8.04	10.24	14.71	+ .36
	9.00	11.80	16.54	+ .24
	12.38 P. M.	15.13	20.30	+ .12
	7.03	7.07	11.58	+ .02
	8.08	6.72	11.02	+ .20
	8.58	6.38	10.71	+ .13
	10.16	5.04	9.52	— .14
October 6th.....	4.32 A. M.	3.51	7.58	+ .12
	5.28	3.47	7.75	— .09
	6.28	4.76	9.01	+ .06
	7.56	9.59	14.23	+ .13
	8.54	11.95	16.78	+ .16
October 8th.....	7.54	8.62	13.50	— .20
	8.48	10.89	16.02	— .24
	9.52	12.55	17.96	— .36
	10.46	13.60	18.56	+ .19
	11.52	14.63	19.98	— .11
	12.58 P. M.	15.02	20.24	+ .06
	2.06	15.63	20.77	+ .20
	3.00	14.67	19.76	+ .16
	4.06	13.87	18.77	+ .27
	5.52	9.45	14.27	— .07
	7.12	7.38	11.83	+ .11
	11.16	7 25	12.04	— .24
	11.58	7.70	12.48	— .19
October 9th.....	2.04 A. M.	5.31	9 80	— .13
	3.04	5.49	10.05	— .18
	4.08	5.49	9.97	— .10
	5.06	4.19	8.76	— .29
	6.10	4.47	8.95	— .20
	6.56	6.68	11.38	— .21
	8.00	9.33	14.22	— .15

TABLE No. 2.

Results of Observations for Length and Rate of Expansion of 100-Meter Steel Tape No. 88.

Date.	Time of Day.	Temperature of Tape.	Observed Length of Tape.	Residual (computed minus observed value).
1891.				
August 1st.....	10.14 A. M.	27.46° C.	$20B_{17} + 35.86^{mm}$	$+ 0.42^{mm}$
	1.52 P. M.	29.28	38.66	— .40
	2.50	24.96	32.94	+ .61
	3.32	24.61	33.48	— .31
	4.10	22.86	31.35	— .09
	4.56	23.34	32.41	— .63
	5.56	23.97	31.89	+ .58
	7.25	20.46	28.17	+ .46
August 3d..... ...	7.52 A. M.	14.99	22.73	— .06
	8.29	14.46	21.83	+ .26
	9.59	14.30	22.57	— .65
	11.05	14.70	21.96	+ .39
	1.32 P. M.	16.45	24.17	+ .09
	2.08	16.95	24.79	+ .02
August 4th	8.22 A. M.	20.84	29.12	— .07
	9.02	21.66	29.82	+ .13
	10.02	23.24	31.59	+ .08
	10.56	24.14	32.91	— .26
	12.02 P. M.	24.01	32.87	— .37
	1.02	25.01	33.60	+ .00
	4.46	24.77	33.03	+ .31
	9.04	17.72	25.16	+ .49
August 5th.....	7.15 A. M.	21.71	29.95	+ .05
	8.32	24.29	32.50	+ .32
	9.32	25.28	33.47	+ .43
	10.32	25.71	33.83	+ .54
	11.32	28.01	37.08	+ .21
	1.32 P. M.	28.01	36.99	+ .12
	2.32	28.27	37.03	+ .13
	3.32	27.89	36.60	+ .15
	4.30	26.31	34 74	+ .28
	5.32	26.27	34.88	+ .10
	8.56	17.67	25.06	+ .54
August 30th	6.46 P. M.	16.41	23.93	+ .29
	8.14	14.26	21.49	+ .39
	9.30	13.15	20.65	+ .01
September 5th.....	12.08 P. M.	14.55	22.58	— .39
	1.16	15.10	23.36	— .57
	1.42	15.78	24.09	— .56
	2.24	15.28	23.49	— .50
	3.45	14.28	21.96	— .06
	4.54	14.26	22.02	— .15
October 3d	10.55 A. M.	27.08	36.31	— .46
	11.39	26.99	36.44	— .67
	1.49 P. M.	26.93	36.44	— .74
October 4th.....	4.44	13.83	21.94	— .54
	7.02	12.60	20.16	— .10
	8.01	12.30	19.98	— .25
	9.10	11.53	18.92	— .03
	10.10	11.50	18.86	— .10
October 5th.....	5.12	7.03	14.15	— .18
	5.56	6.46	13.66	— .30
	7.00	8.25	15.37	— .06
	7.56	9.97	16.90	+ .29
	9.04	11.70	18.85	+ .23
	12.30 P. M.	15.35	23.05	+ .01
	7.12	7.02	13.71	+ .25
	8.02	6.42	13.27	+ .05
	9.08	6.54	13.42	+ .03
	10.04	5.12	12.03	— .13

TABLE No. 2—*Continued.*

Date.	Time of Day.	Temperature of Tape.	Observed Length of Tape.	Residual (computed minus observed value).
October 6th.....	4.42 A. M.	4.39° C.	$20B_{17} + 11.20^{mm}$	$-.10^{mm}$
	5.22	3.46	10.13	— .04
	6.34	4.66	11.29	+ .11
	7.48	8.82	16.12	— .18
	9.02	12.70	20.01	+ .16
October 8th.....	7.50 A. M.	8.15	15.57	— .36
	8.54	11.20	18.57	— .04
	9.48	12.14	19.44	+ .16
	10.52	13.85	21.24	+ .19
	11.46	14.05	21.10	+ .54
	1.04 P. M.	15.25	22.81	+ .14
	2.00	16.12	23.55	+ .35
	3.06	14.62	22.21	+ .06
	4.02	14.22	21.73	+ .10
	5.58	9.23	16.08	+ .20
	7.06	7.57	14.44	+ .13
	11.06	7.35	14.67	— .34
October 9th.....	12.08 A. M.	7.52	14.71	— .19
	1.56	5.39	12.57	— .38
	3.10	5.29	12.04	+ .04
	4.04	5.67	12.56	— .06
	5.12	4.21	10.86	+ .04
	6.04	4.11	10.84	— .04
	7.02	7.07	14.25	— .22
	7.56	9.10	16.71	— .47

Derived Values and Probable Errors.—Assigning equal weights to the observations in the above tables, the method of least squares gives the following equations for the tapes:

$$T_{85} = 20B_{17} + 22.35^{mm} \pm 0.023^{mm} + (1.0947^{mm} \pm 0.0025^{mm})\ (t - 16.89° \text{ C.}) \quad \ldots\ldots (1)$$

$$T_{88} = 20B_{17} + 23.47^{mm} \pm 0.022^{mm} + (1.0914^{mm} \pm 0.0029^{mm})\ (t - 15.72° \text{ C.}) \quad \ldots\ldots (2)$$

B_{17} = length of steel bar No. 17 in ice = $5^m - 16^\mu$ very nearly,

t = temperature of tape in degrees C.,

16.89° C. = mean of observed temperatures of tape No. 85,

15.72° C. = mean of observed temperatures of tape No. 88.

The probable errors of the mean lengths of the tapes and of their rates of expansion given above are derived in the following manner: The sum of the squares of the residuals for—

tape No. 85 = 4.82,

tape No. 88 = 8.62;

and since the number of residuals in the first case is 77 and in the other 85, the probable error of a single determination of length for—

tape No. 85 = $\pm 0.17^{mm}$,

tape No. 88 = $\pm 0.22^{mm}$.

Since there is no *a priori* reason for attributing any greater weight to the observations on tape No. 85 than to those on tape No. 88, the mean of these two values, or $\pm 0.20^{mm}$, is adopted as the probable error of a single determination of a tape length on the 100^{m} comparator. This error is $\frac{1}{500000}$ part of the tape's length. It is due almost wholly to imperfect temperature indications of the thermometers, since the means for referring the ends of the tape to the fiducial stones of the comparator would admit errors of a few hundredths of a millimeter only.

The weights of the mean lengths and the rates of expansion which result from the least square solution are for—

tape No. 85, 77 and 6 436,
tape No. 88, 85 and 4 687.

These, in connection with the probable error of a single determination of length derived above, give the probable errors attached to the quantities in equations (1) and (2).

Without attempting a full discussion of these results here, it may be stated that the above tables include every determination made of the tape lengths in question. The temperature range is from 3.5° C. to 32.1° C. for No. 85, and from 3.5° C. to 28.0° C. for No. 88. During the period covered by the observations, August 1st to October 9th, the tapes were wound on and unwound from their reels many times and used frequently to measure the different sections of Holton Base.

The maximum residual in the comparisons of tape No. 85 is 0.70^{mm}, and the maximum for No. 88 is 0.74^{mm}. These correspond to $\frac{1}{140000}$ part, about, of a tape length. The following table gives a comparison between the actual distribution of the whole number of residuals (162 for both tapes) and the theoretical distribution computed by means of the law of error of least squares.

TABLE No. 3.

Distribution of Residuals.

Between Limits.	Actual Number of Residuals.	Theoretical Number.
0.0^{mm} and 0.2^{mm}	85	84
0.2 " 0.4	51	52
0.4 " 0.6	17	21
0.6 " 0.8	9	5

An inspection of the residuals in Tables Nos. 1 and 2 indicates that some lagging of the tape's temperature relatively to that of the thermometers occurred at times.

For practical applications the equations (1) and (2) may be written in the following more compact form:

$$T_{85} = 20B_{17} + 3.86^{mm} + 1^{mm}.0947\,t \ldots\ldots\ldots\ldots\ldots (3)$$
$$T_{88} = 20B_{17} + 6.31^{mm} + 1^{mm}.0914\,t \ldots\ldots\ldots\ldots\ldots (4)$$

These equations give the lengths of the tapes when they are under a tension of 11.595 kg. (25 lbs. 9 ozs.), and when supported at equidistant intervals of 10^m.

The probable errors of the mean lengths in equations (1) and (2) are due to the errors of comparison only. The total probable errors of those mean lengths, when expressed in meters, depend, 1st, on the errors of comparison just mentioned; 2d, on the error in length of the 100^m comparator as measured with the iced bar; 3d, on the error in length of the iced bar when expressed in terms of the International meter.* The second and third errors when expressed as ratios may be taken as not greater than $\frac{1}{3000000}$ and $\frac{1}{4000000}$ part, respectively, or in 100^m, as not exceeding $\pm\ 0.020^{mm}$ and $\pm\ 0.025^{mm}$ respectively. These combined with the probable errors $\pm\ 0.023^{mm}$ and $\pm\ 0.022^{mm}$, in equations (1) and (2), give $\pm\ 0.039^{mm}$ as the probable error of either tape expressed in meters at the mean of its observed temperatures on the 100^m comparator. For such extreme temperatures as 0° C. and 30° C., which are rarely reached in actual work, the probable errors rise to—

$\pm\ 0.057^{mm}$ and $\pm\ 0.052^{mm}$ respectively for tape No. 85, and
$\pm\ 0.060^{mm}$ and $\pm\ 0.057^{mm}$ respectively for tape No. 88.

The square root of the average of the squares of the probable errors of the tape lengths between 0° and 30°, supposing all such temperatures of equal frequency, is $\pm\ 0.045^{mm}$ for tape No. 85, $\pm\ 0.046^{mm}$ for tape No. 88. Hence it is concluded that for such temperatures as are likely to be had in the use of these tapes the probable errors of their absolute lengths will not exceed $\pm\ 0.05^{mm}$ or $\frac{1}{2000000}$ part of their lengths.

* To these errors it may be essential to add the error of reference to the hydrogen scale of the mercurial thermometers used in determining the tape temperatures. It does not appear at present, however, that such error can appreciably increase the probable errors given in the text.

Results Obtained in the Measurement of St. Albans Base.

Explanation of Data.—In order to furnish a fairly complete idea of the precision attainable in the actual work of measuring a base, the results of the measures of St. Albans Base, 1892, will be given herewith. This base of 3 870.5^m was divided into four sections. The first three of these beginning at the west end were closely equal to 1 km. each. Five measures of this base were made. Four of them were made during the night time, when the temperature varied from 5° to 15° C. in the means for the several sections. The fifth measure was made during the day, in full sunshine, at a time when the temperature varied from 27° to 32° C. in the means for the different sections. The first two and the last measures were made with tape No. 88; the third and fourth measures were made with tape No. 85.

The wide range of temperature of these measures, and the use of two tapes whose equations differ, as shown above, by about 2.5^{mm}, would appear to afford a pretty good test of the efficiency of the apparatus and method.

Table No. 4, following, gives the results of these measures. The first column gives the date; the second, the time of day when the measure was made; the third, the direction in which the measure proceeded; the fourth, the mean temperature of the tape; the fifth, the temperature range, or the difference between the highest and lowest observed temperature of the thermometers; the sixth indicates by the letters R and F whether the temperature was rising or falling during the measure of a section, and the order of the letters indicates the order in which the changes occurred; the seventh gives the length of the sections in terms of the iced bar B_{17}, the excess over a round number of tape and bar lengths having been measured with a millimeter scale, and with a 15^m auxiliary steel tape in case of the fourth section; and the last column gives the number of the tape used.

TABLE No. 4.

RESULTS OF MEASURE OF ST. ALBANS BASE.

West Base to Stake 10.

Date.	Time of Day.	Direction of Measure.	Mean Temperature.	Temperature Range.	Temperature Rising, *R*, Falling, *F*.	Value for Distance.	No. of Tape.
1892.	P.M.						
October 11...	7.20- 8.02	W. to E.	9.04° C.	3.40° C.	*F*	$200\,B_{17} + 258.2^{mm}$	88
12...	9.03- 9.44	E. to W.	7.62	2.70	*R*	244.5	88
13...	7.06- 7.40	W. to E.	13.30	3.00	*F*	252.6	85
13...	10.49-11.21	E. to W.	9.56	1.10	*R*	251.5	85
14...	2 53- 3.16	W. to E.	32.11	1.40	*R, F*	248.1	88
			STAKE 10 TO STAKE 20.				
October 11...	8.02- 8.44	W. to E.	5.92° C.	2.80° C.	*F, R*	$200\,B_{17} + 534.8^{mm}$	88
12...	9.44-10.25	E. to W	6.39	2.25	*F, R*	535.5	88
13...	7.40- 8.12	W. to E.	10.38	2.55	*F, R*	533 0	85
13...	11.21-11.53	E. to W.	9.05	0.80	*F, R, F*	536.0	85
14...	3.16- 3.49	W. to E.	30.47	0.80	*F, R*	536.0	88
			STAKE 20 TO STAKE 30.				
October 11...	8.44- 9.26	W. to E.	6.23° C.	1.50° C.	*R, F, R*	$200\,B_{17} + 527.0^{mm}$	88
12...	7.40- 8.22	E. to W.	7.84	1.60	*R, F*	524.0	88
13...	8.12- 8.44	W. to E.	9.92	1.10	*F, R, F*	527.8	85
13...	9.45-10.17	E. to W.	8.75	0.70	*R, F, R*	528.9	85
14...	3.49- 4.02	W. to E.	30.80	1.50	*R, F, R*	534.0	88
			STAKE 30 TO EAST BASE.				
October 11...	9.36-10.08	W. to E.	4.80° C.	0.75° C.	*F, R*	$172B_{17}+9\,325.5^{mm}$	88
12...	8.22- 9.03	E. to W.	7 92	2.65	*F, R*	9 323.0	88
13...	8.44- 9.16	W. to E.	9.24	0.50	*F, R*	9 321.4	85
13...	10.17-10.49	E. to W.	8.57	1.00	*F, R, F*	9 328.6	85
14...	4.02- 4.26	W. to E.	27.46	0.95	*R, F, R*	9 333.0	88

Interpretation of Results.—Without entering into a minute discussion of these results, attention may be called especially to what is considered the key to their proper interpretation. In case there is any relative lagging of the tape and thermometric temperatures, it is obvious that the effect of such lagging will be more or less eliminated in the mean of two measures of a section made, the one under a rising and the other under a falling temperature. A similar elimination of the lag effect will occur in a single measure of a section if the temperature is rising for one half and falling for the other half, or *vice versa*, of the measure; and in general, the lag effect will tend to eliminate itself if the temperature fluctuates to any considerable extent about its mean trend.

These considerations seem to afford a satisfactory explanation of the ranges among the results of the measures of the several sections.

Thus, as might be expected from these considerations in connection with the data shown in the fifth and sixth columns of the above table, the range is greatest among the measures of the section from West Base to stake 10. The range is least in the measures of the section from stake 10 to stake 20. The middle portion of this latter section was on low, marshy ground compared with the ground at either end; so that, as indicated by the letters in the sixth column of the table, the temperature conditions were favorable to an elimination of the lag effect in every measure of the line.

Each section of the base was measured at night twice, once in each direction, by each of the two tapes. The conditions as to temperature, etc., differed little during the three nights when these measures were made ; *i. e.*, the sky was cloudless, or nearly so, there was little or no wind, and the measures were made while dew was condensing along most parts of the base line. Owing to these conditions, and the topographic conformation of the ground along the line, the temperature trend was in general reversed by the reversal of the direction in which the measures proceeded. For this reason we should expect the means of forward and backward measures of the sections to be largely free from the effect of lagging of the thermometers. This appears to be the case as shown by the following table, which gives the excesses in millimeters of those means for each section as measured by each of the two tapes :

TABLE No. 5.

MEANS OF FORWARD AND BACKWARD MEASURES OF THE SECTIONS.

No. Tape.	Section 1.	Section 2.	Section 3.	Section 4.	Whole Base.
88	$+251.4^{mm}$	$+535.2^{mm}$	$+525.5^{mm}$	$+9\,324.2^{mm}$	$10\,636.3^{mm}$
85	252.0	534.5	528.4	9 325.0	10 639.9

The accordance of these results leaves little to be desired ; but for the reasons already adduced, and for others given below, it is difficult to see how it can be otherwise than real.

Although the fact that means of forward and backward measures of a section indicated a marked elimination of some source of error was noticed in the measures of Holton Base in 1891, the explanation of that fact, presented above, did not occur to me until after the night measures

of St. Albans Base had been made. The full import of the explanation, indeed, was not appreciated until after the day measures of October 14th had been made; otherwise a reverse measure of the line would have been made on the latter day, though, as it happened, the temperature conditions were fairly favorable on that date for eliminating the lag effect.

In deriving the total length of the base, it is considered best to use the night measures only, since all our experience shows that the temperature uncertainty is least in such measures. Accordingly, the value adopted is—

$$772\,B_{17} + 10\,638.1^{mm}$$

or the mean of the four night measures.

If, on the other hand, we include the day measure and give it and each of the night measures the same weight, the result is—

$$772\,B_{17} + 10\,640.7^{mm}$$

which differs but 2.6^{mm} from the adopted value.

The results for the several measures of the whole base, in the order in which they were made, are—

$$772\,B_{17} + 10\,645.5^{mm}$$
$$27.0$$
$$34.8$$
$$45.0$$
$$51.1$$

The range among these is 24.1^{mm}, or $\frac{1}{160000}$ part of the whole line. The range among the first four, or the night measures, is 18.5^{mm} or $\frac{1}{209000}$ part. The greatest divergence from the adopted mean is that of the last or day measure. The amount is 13^{mm}, or $\frac{1}{298000}$ part.

Probable Errors.—In computing probable errors from the above data we may proceed according to several hypotheses. Using 1 km. as the unit of distance to which such errors refer, we may assume (*a*) that all of the night and day measures of the several sections of the base are of equal weight, and that no elimination of systematic error results in the mean of a forward and backward measure; (*b*) that the night measures should be considered by themselves and treated as if they involved no errors due to lagging of the thermometers; (*c*) that the means of forward and backward measures of the sections as given in Table No. 5 are more or less free from errors due

to lagging, and that such means are alone competent to give an idea of the attainable precision.

The errors which result from each of these hypotheses will now be given. Let ε_a, ε_b, ε_c = the probable error of one measure of a kilometer according to the suppositions (a), (b), (c), respectively.

m = the total number of independently observed quantities, or total number of measures of the sections,

$= 20$ for supposition (a),
$= 16$ " " (b),
$= 8$ " " (c),

$[vv]$ = sum of squares of the discrepancies between the individual measures of the sections and their means, respectively.

Then from Tables Nos. 4 and 5 we find*

$[vv] = 250.57$ for supposition (a),
$= 142.94$ " " (b),
$= 4.80$ " " (c).

According to the method of least squares, when we have m measures of equal weight of μ unknown quantities, the probable error of a single measure is expressed by—

$$\pm\ 0.6745\ \sqrt{\frac{[vv]}{m-\mu}}$$

In the present case the number of unknown quantities, or $\mu = 4$, the number of sections of the base. Hence for the several suppositions we have $m - \mu = 16, 12, 4$, respectively. These data then give—

$$\varepsilon_a = \pm 2.67^{mm}$$
$$\varepsilon_b = \pm 2.33$$
$$\varepsilon_c = \pm 0.74$$

It may be observed that if the mean values in Table No. 5, from which ε_c is derived, are not to a marked extent free from systematic error, as is assumed, we should have by the theory of errors—

$$\varepsilon_c = \tfrac{1}{2}\ \varepsilon_b\ \sqrt{2} = 0.71\ \varepsilon_b.$$

But the actual relation is $\varepsilon_c = 0.32\ \varepsilon_b$, or the ratio of the two errors is less than half as great as it should be if the hypothesis (c) were false.

It may be observed, also, that the work on the long comparator in determining the tape lengths affords an indirect means of estimating the precision attainable in the mean of two measures of a kilometer.

* The sections are treated as of equal length, the modification arising from the fact that the section from Stake 30 to East base is 130^m short of a kilometer being unimportant.

As shown above, the probable error of a single determination of a tape length on that comparator is $\pm 0.20^{mm}$. This, as already explained, is almost wholly due to temperature uncertainty in the mean of six thermometer readings. If, now, we assume that the temperature uncertainty is on the average no greater in night work on a base line, and it does not appear to be so if the measures are made in a way to eliminate the lag effect, the probable error of a tape length from the mean of two in place of six thermometer readings should be $\pm 0.20^{mm} \sqrt{\frac{6}{2}} = \pm 0.20^{mm} \sqrt{3}$. With this for the probable error of a single tape length, the probable error of the sum of ten such lengths, or of one kilometer, is $\pm 0.20^{mm} \sqrt{30}$; and hence the probable error of the mean of two such measures is—

$$\pm 0.20^{mm} \sqrt{15} = \pm 0.77^{mm},$$

which does not differ materially from the value $\varepsilon_c = \pm 0.74^{mm}$ derived above. While no great importance is attached to this argument, since the results in Table No. 5 are too few to render such argument conclusive, it seems not out of place to note that the precision attained in this field work is not inconsistent with that attained in determining the tape lengths.

Lastly, it may be observed that the value of $\varepsilon_b = \pm 2.33^{mm}$, derived by using the night measures alone of St. Albans Base, is somewhat larger than the corresponding value which results from the measures of the sections of Holton Base made with the same tapes in 1891. Disregarding the measures of the two short sections (one of 100^m and one of 500^m) of this base, measures presenting abnormally small ranges, the 41 night measures of the remaining five sections, which vary in length from 900^m to $1\,200^m$, give for ε_b, or the probable error of one measure of a kilometer, $\pm 1.88^{mm}$. This smaller value is probably due partly to the smaller amount of lagging of the thermometers as fitted with the aluminum sheaths in 1891, and partly to the exceptionally favorable conditions of the standard kilometer of Holton Base for eliminating the lag effect. This section had about 200 meters at either end in open fields, while the intervening 600 meters were along a narrow lane cut through a dense forest. By reason of these conditions there was in general a marked change in temperature in passing from field to forest and *vice versa*, which change, as already explained, is favorable to the elimination of the lag effect in any single measure.

The 21 night measures of this kilometer taken by themselves show a probable error of $\pm 1.63^{mm}$ for one measure.

Now, as to the probable error, due to the errors of measurement just considered, in the whole length, or sum of the sections, of St. Albans Base, three values dependent on the three suppositions (*a*), (*b*), (*c*), respectively, may be derived. Thus, the probable error of the whole base is—

$$\varepsilon_a \sqrt{\frac{4}{5}} = \pm 2.39^{mm} \text{ according to supposition } (a),$$

$$\varepsilon_b \sqrt{\frac{4}{4}} = \pm 2.33 \quad \text{“} \quad \text{“} \quad (b),$$

$$\varepsilon_c \sqrt{\frac{4}{2}} = \pm 1.04 \quad \text{“} \quad \text{“} \quad (c).$$

Expressed as fractions of the base, these vary from the $\frac{1}{162000}$ to the $\frac{1}{372000}$ part.

The errors just discussed, or those developed in repeated measures of a line, appear to be the only ones of the compensating class which need be discussed. In comparison with these, the errors in grade corrections, in transfers to permanent marks, in the measurement of small fractions of tape lengths, etc., are trifling. It would seem safe to conclude, therefore, that the probable error of the adopted length of St. Albans Base, arising from all sources save that of error in the tape lengths, cannot exceed $\frac{1}{2000000}$ part.

With respect to the probable error of a tape length, the results of the determinations on the Holton comparator indicate that it cannot on the average exceed $\pm 0.05^{mm}$ as already explained. One circumstance, however, leads me to adopt a larger value. It is this: if we use the mean lengths of the sections of St. Albans Base given in Table No. 5, to compute a correction to the assumed difference in length of the tapes at 0° C., the mean value of such correction is found to be $+0.09^{mm} \pm 0.095^{mm}$, indicating that the difference in length of the tapes at 0° C. should be 2.54^{mm} instead of 2.45^{mm} as given by equations (3) and (4). Subseqently to the measurement of the base a series of comparisons of the tapes was made, mostly during the day and in sunshine, on a 100^{m} section of the base. These comparisons indicate a correction of the same amount with somewhat greater certainty. The amount of this discrepancy, it will be observed, is of about the same

magnitude as the probable error of the difference of the tape lengths derived from equations (1) and (2), viz.:

$$\sqrt{(0.057^{mm})^2 + (0.060^{mm})^2} = \pm 0.08^{mm},$$

so that no great weight can be attached to it. But inasmuch as the lengths of the tapes were determined by observing on their graduations with micrometer microscopes, while in applying them to measure a base a hand magnifier or the unaided eye is used, it is possible that some systematic error may have entered the measures of St. Albans Base. For this reason the extreme probable error of a tape length given on the 15th page of this paper, namely, $\pm 0.06^{mm}$, will be adopted in computing the probable error of this base. Combining this with the probable error from other sources of error, estimated above as not exceeding $\frac{1}{2000000}$ part, or $\pm 0.05^{mm}$ per tape length, we have for the total probable error of St. Albans Base—

$$\pm 3.02^{mm}, \text{ or } \frac{1}{1280000} \text{ part.}$$

The adopted length of the base as measured may then be written—

$$772\, B_{17} + 10\,638.1^{mm} \pm 3.0^{mm}.$$

To this must be applied a correction of -112.7^{mm} for grades, or slopes of the tape lengths, and a correction of -109.1^{mm} for an elevation of 180^{m} above sea level.

MATHEMATICAL APPENDIX.

Formulas Relative to Length of Tape.

Actual and Apparent Tension by Spring Balances.—If the spring balance used to give tension to the tape is adjusted to give correct tensions when in a vertical position, it will indicate (by its face readings) less than the actual tension when in a horizontal position. Such balances are liable also from wear and other causes to have an appreciable index correction.

To determine the actual tension in any case with such balances, let r be the index correction. It is the reading of the index when the balance is vertical, hook end down, and without load on the hook; it is *minus* when the index reads greater than zero, and *plus* when it reads less. Let R be the reading of the index when the balance is suspended by its hook, hook end up. Let W be the total weight of the balance found by weighing it on another balance. Then if T_1 denote the ob-

served or face reading of the balance when horizontal, and T denote the corresponding actual tension—

$$T = T_1 + r + \tfrac{1}{2}(W - R - r)$$
$$= T_1 + \tfrac{1}{2}(W - R + r).$$

Example.—On September 25th, 1891, the following values were observed with balance No. 53, U. S. C. & G. S., namely ·

$r = -$ 3.0 ozs.,
$R =$ 35.0 "
$W =$ 49.2 "
$T =$ 25.0 "

Hence—

$$T = 25 \text{ lbs.} + 5.6 \text{ ozs.}$$

Change in Length of Tape due to Change in Tension, Omission of One or More Supports, etc.—When the tension applied to a tape differs from the normal tension, or the tension under which the tape was standardized, a change in length of the tape results. Likewise a change in length will occur when the interval between equi-distant supports of the tape is changed, or when some of the supports are omitted. It is often convenient to omit one or more supports, as in crossing a road, a ravine, etc. Not infrequently, also, tapes are standardized by laying them on a flat or mural standard, and it thus becomes essential to know the shortening they undergo when supported at finite intervals instead of throughout their length. The following formulas apply to these cases :

Let—

w = the weight per unit length of the tape,
τ = the applied tension,
$a = \dfrac{w}{\tau}$
n = the number of sections into which the tape is divided by equi-distant supports,
l = the length of any such section,
L = the normal length of the tape, or the right line distance between its end marks when under standard tension,
$= \Sigma\, l = nl$, approximately,
μ = the reciprocal of the product of the modulus of elasticity of the tape by the area of its cross-section.

(1) Then the change in length, ΔL_1 say, in L due to a change $\Delta\tau$ in the tension is—

$$\Delta L_1 = n\, l\, \mu\, \Delta\tau + \tfrac{1}{12}\, a^2\, n\, l^3\, \frac{\Delta\tau}{\tau} \ldots\ldots\ldots\ldots\ldots (1)$$

Example.—For tape No. 85, when supported at equi-distant intervals of 10^m we have—

$$\begin{aligned} n &= 10, \\ l &= 10^m, \\ w &= 22.32 \text{ grams per meter}, \\ \tau &= 25.5 \text{ lbs.} = 11\,566.66 \text{ grams}, \\ a^2 &= 372 \times 10^{-8}, \\ \mu &= 16 \times 10^{-9} \text{ for gram as unit*} \\ &= 450 \times 10^{-9} \text{ for ounce as unit.} \end{aligned}$$

Hence, for $\Delta\tau = 1$ oz.,

$$n\, l\, \mu\, \Delta\tau = 10 \times 10^m \times 450 \times 10^{-9} = 0.045^{mm},$$

$$\frac{1}{12}\, a^2\, n\, l^3\, \frac{\Delta\tau}{\tau} = \frac{1}{12} \times 372 \times 10^{-8} \times 10^7 \times \frac{1}{408} = 0.0076^{mm}.$$

$$\Delta L_1 = 0.0526^{mm}.$$

It may be observed that the quantity ΔL_1 can be measured directly by increasing and decreasing the tension in the vicinity of any assumed value of τ. The mean of several values determined in this way, with tape No. 85, is $\Delta L_1 = 0.053^{mm}$ per ounce when $\tau = 25.5$ lbs.

(2) Suppose a given tape to be divided by its equi-distant supports, 1st, into n_1 sections of length l_1; and 2d, into n_2 sections of length l_2. Then assuming the tension the same in both cases, if $n_2 > n_1$, the difference in distance between the end graduations of the tape, ΔL_2 say, in the two cases, will be expressed by—

$$\Delta L_2 = \Sigma\,(l_2 - l_1) = \frac{1}{24}\, a^2\, (n_1\, l_1^3 - n_2\, l_2^3) \ldots\ldots\ldots\ldots (2)$$

Example.—For the 100^m tape No. 85, $n_1 = 5$ and $l_1 = 20^m$ when the supports are 20^m apart; and $n_2 = 10$ and $l_2 = 10^m$ when the supports are 10^m apart. Hence, in this case if $a^2 = 372 \times 10^{-8}$, as above—

$$\Delta L_2 = \frac{1}{24} \times 372 \times 10^{-8} \left(5\,(20)^3 - 10\,(10)^3\right) = 4.65^{mm}.$$

If a single support is omitted, we have only to make $n_2 = 2$, $n_1 = 1$, and $l_1 = 2\, l_2$ in equation (2). Thus the omission of a single support

* Since the cross-section of the tape is $6.34^{mm} \times 0.47^{mm}$, or 0.0298 sq. cm., the value of μ corresponds to a modulus of 2.1×10^6 kilos. per square centimeter, or 30×10^6 lbs. per square inch.

when $n_2 = 10$ and $l_2 = 10^m$, shortens tape No. 85 by 0.93^{mm}, supposing the normal tension to be 25.5 lbs. as above.

Similarly, the omission of m consecutive supports shortens a tape by—

$$\frac{1}{24} m (m+1)(m+2) a^2 l^3,$$

where l is the length of a section when no supports are omitted.

(3) When a tape is supported throughout its length, as when lying on a horizontal plane or on a mural standard, n_2 in formula (2) becomes infinite and l_2 infinitesimal. If we call the length of the tape in this case $L_o = \Sigma l_2$, and omit the suffixes from n_1 and l_1, formula (2) gives—

$$L_o - \Sigma l = \frac{1}{24} a^2 n l^3 = \Delta L_3, \text{ say} \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

This formula shows, for example, that tape No. 85 when supported at equi-distant intervals of 10^m, and when under a tension of 25.5 lbs., is shorter by 1.55^{mm} than it would be when under the same tension on a horizontal surface.

(4) Lastly, it will be of interest to have an expression for the change in length of a tape due to a change in its weight per unit length. Such a change may occur by reason of a deposit of dew on the tape or from wear of the same. Let Δw be the change per unit length, or the increment to w; and let the corresponding change in the total length of the tape be denoted by ΔL_4. Then, assuming as above that the number of sections of the tape is n, and that their lengths are each l approximately—

$$\Delta L_4 = -\frac{1}{12} n l^3 a^2 \frac{\Delta w}{w} \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

This formula shows, for example, that when tape No. 85 is supported at equi-distant intervals of 10^m and is under a tension of 25.5 lbs.; or when—

$$l = 10^m,$$
$$n = 10,$$
$$a^2 = 372 \times 10^{-8},$$
$$\Delta L_4 = -3.1^{mm} \times \frac{\Delta w}{w}.$$

Thus, in order to produce a change of $\frac{1}{1000000}$ part in the tape's length, or in order that ΔL_4 may be 0.1^{mm}, we must have $\Delta w = \frac{w}{31}$.

Formulas for Computation of Length of Measured Line.

Summation of Tape Lengths and Correction for Temperature.—The formulas of the preceding section give the actual length of the tape in

terms of its standard length, the index correction to the balance, etc. Such actual or working length may be most conveniently expressed in the form—

$$L = A + a + \alpha t,$$

wherein A is the nominal length of the tape, a round number of meters or feet; a is a small excess over that round number; α is the rate of expansion of the tape; and t is the tape's temperature in degrees C. Then the sum of N such lengths, t being supposed to vary from one length to another, will be—

$$\Sigma L = NA + Na + \alpha \Sigma t. \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

The last term in this expression is the correction for temperature of the tape. As here given it assumes that the t's have been corrected for errors of the thermometers used. Generally, it suffices to apply an average correction to the observed values of t for several tape lengths, since the temperature does not often change much during the measurement of a section of a base.

Corrections for Set-Ups and Set-Backs and End Corrections.—These quantities, which are usually very small, are measured with a millimeter or other suitable scale. Their application in computing a measured length is obvious.

Correction for Grades.—The correction for grade is computed for each tape length, such length having a uniform slope from end to end. The difference in altitude of the ends of the tape, or the difference in altitude of the corresponding marking tables, is measured with an engineer's level. Call this difference h. The length of the tape being L, its horizontal projection will be—

$$\sqrt{L^2 - h^2};$$

and if we call the grade correction c,

$$c = L - \sqrt{L^2 - h^2}$$
$$= -\left(\frac{h^2}{2L} + \frac{h^4}{8L^3} + \ldots\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Usually the first term of this series suffices when a long tape is used. Thus, for a 100^m tape—

$$\frac{h^4}{8L^3} < 0.01^{mm} \text{ for } h < 2.9^m,$$

so that for such a tape, in all ordinary cases—

$$c \text{ in millimeters} = 5\ (h \text{ in meters})^2.$$

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

637.

(Vol. XXX.—October, 1893.)

TERRESTRIAL MAGNETISM IN NORTH AMERICA.

By CHARLES A. SCHOTT, Assistant U. S. Coast and Geodetic Survey.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

In this paper it is proposed to give, first, an historical outline from the time of Columbus to our own, showing the gradual accessions to our knowledge of terrestrial magnetism, with special reference to North America, to be followed by an exposition of, or reference to, the numerous laws so far recognized, with more or less certainty, as governing the changes in the direction and intensity of the earth's magnetic force within the specified region and period. This limitation is essential and will allow more space for the presentation of matter which may be of special interest to Americans.

The subject under consideration commends itself to our attention, both from a scientific and a practical point of view. The latter relation is obvious, since we have but to point to the latest evolution in engineers' activity, viz., electrical engineering. Of the former, it is

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

proposed here to give an account of the distribution of magnetism in its three elements—declination, dip and horizontal force—as well as of the continual changes, periodic or secular, to which they are subject. This subdivision is a matter of convenience for the analysis as well as for the instrumental measure of the magnetic force. The declinometer (or compass) determines the direction of the force in the horizontal plane; the inclinometer, or dip circle, the direction in a vertical plane of a freely suspended magnetic needle; and the magnetometer gives, in absolute measure, generally expressed in units of the C. G. S. system, the intensity of the force or of its horizontal component; hence, also, the vertical component and the total intensity.

Terrestrial magnetism considered as a science had no existence in the days of Columbus, and but little more could be gathered, in his time, of magnetic phenomena than the ancient knowledge of the attraction of the lodestone for iron and its directive force. The dual character of the magnetic property was apparent enough; that in the southwestern part of Europe the needle (north-seeking end) pointed slightly to the eastward was recognized, and it was believed that the declination at any place was invariable.

Not to refer to the current fable of a magnetic mountain in the north as the cause of attraction, even Galileo, at a much later period, thought that the fixed direction of the earth's axis might be due to magnetic attraction of a point in space.

Yet we owe to Columbus the discovery of a point in the agonic curve of the North Atlantic where, on September 13th, 1492, in latitude $+28^\circ\ 21'$ and in longitude $29^\circ\ 16'$ west from Greenwich, he noticed that the variation had changed from the east to the west side of the meridian. The declination remained westerly to the end of his voyage of discovery. He was closely followed by Sebastian Cabot (1497–98), who found, farther north and 110 miles west of Flores (Azores),* a second point of the same agonic. The declination along the Atlantic coast of North America, north of latitude 40° (about), has remained westerly to this day, though south of this parallel it has changed to *easterly*.

A century after Columbus, Dr. Gilbert† (styled the father of the

* C. & G. S. Bulletin, No. 6 (May, 1888).

† Appendix No. 19, C. & G. S., Report for 1880.

magnetic philosophy) published in 1600, his celebrated work, "De Magnete." He showed the earth to be a great magnet and elucidated most of the fundamental facts of magnetism. Thus we owe to him the terms "magnetic poles," "axis," "magnetic meridians," as well as "magnetic latitudes" (as measured by the dip or by changes in intensity).

The magnetic dip was unknown to Columbus, but was discovered by Hartmann in the year 1544*, though he was unable to measure it; so that the discovery is generally ascribed to Norman, in the year 1576, and was published in his work, "The New Attractive." Unlike declination observations which continued to be made, as of immediate necessity to the seaman, dip observations were only exceptionally noted until the last quarter of the eighteenth century, after which they become more numerous. A far more difficult problem to solve than that involving merely direction was how to measure the intensity of the magnetic force? The feebleness of the force in the earth's equatorial region as compared with its value in the polar regions, where it is more than double the equatorial value, was known to Gilbert (and it was first experimentally proved by Lamanon, from observations made in 1785–87); but it was not until the last quarter of the eighteenth century that even relative measures were practiced, though earlier attempts had been made to this end by counting the oscillations of the dipping needle (Graham, 1723), or of the horizontal needle (Mallet, 1769).

As was the case with the declination, the invariability of the intensity of the earth's magnetic force was at first believed in; but, when the contrary was recognized, the difficulty of comparing intensities at different places and at different times became apparent, not to mention the further complication of the needle gradually parting with its magnetism.

Humboldt, during his travels in tropical America in the years 1798–1803, noted the position of a point in the magnetic equator in Peru, and for some time intensities were expressed in terms of the intensity at that station. Several systems of measures, relative and absolute, were in use by different nations; but, at the present time, the C. G. S. system, proposed in 1881, has gained general acceptance. We may pass by here the several theories on terrestrial magnetism that were

*Dr. R. Wolf's Taschenbuch, Zurich, 1877.

advanced before the time of Gauss. In a memoir published by him in 1833, he showed how the magnetic intensity could be expressed in absolute measure, and in 1838 appeared Gauss' "Theory of Terrestrial Magnetism."* This theory is styled by the late Sir G. B. Airy "one of the most beautiful and the most important that has appeared for many years in physical mathematics."

In this treatise the general theory of terrestrial magnetism is developed independently of any particular hypothesis as to its nature and distribution, but the magnetic force at any place is supposed due to the collective action of all the magnetic particles of the earth's mass. After developing the magnetic potential as a function of three variables in space, he determines its value for any point of the surface and develops the expressions for the three components of the magnetic force, whence for any place the declination, dip and intensity may be calculated. Magnetic data observed at but 12 points were employed, and the 24 coefficients needed to include magnitudes of the fourth order of development were calculated. The treatise is accompanied by a series of isomagnetic charts. Notwithstanding the paucity of observations, the irregularity of their distribution in space, and their want of simultaneity (the epoch being approximately 1830), this first attempt was a signal success. The reduction was repeated with better data in 1874 by Erman and Petersen; in 1881 by Icilius, and in 1885, with greatly extended material, by Dr. Neumayer. These authors confine themselves to the same order of terms as Gauss, and there seems to be no immediate need to include those of the fifth order (or 35 in all). What appears to be of more importance in a new attempt is the elimination of the local irregularities by excluding from the data all stations thus affected. Staff Commander Creak, R. N., pointed out some remarkable regions of disturbances; among these, the Bermuda Islands, likewise the Hawaiian group. But the greatest deviation of the needle due to local causes was recently discovered at a place off the east coast of Australia. It would seem that the magnetism in excess of the normal at these local foci is of the same character as that of the pole of the respective magnetic hemisphere.

Local disturbances were particularly studied and charted by Rücker and Thorpe in connection with their magnetic survey of the British

* For an English translation of it see Taylor's "Scientific Memoirs," Volume II. London, 1841.

Isles in 1886. In the United States, Kentucky apparently shows local deflections on a large scale, but these and a large number of smaller districts of disturbances remain yet to be developed or recognized. Gauss' theory as dependent on observations proves that there are but two magnetic poles on the globe, and there could not be any odd number. Halley's conception of four poles was announced at a time when but a few observations of the declination and inclination and none at all of the intensity had been made, and is obscured by confounding poles with foci of maximum intensity, the latter in the Northern hemisphere being two large regions, one (the stronger) west of Hudson Bay, the other in Central Siberia. These foci seem to be in some way related to regions of maximum cold.

Among the best known and comprehensive isomagnetic charts we mention Hanstein's isogonic charts for the epochs 1600, 1700 and 1800; Edm. Halley's "Tabula Nautica," anno 1700,* comprising mainly the Atlantic and Indian oceans. The Philosophical Transactions of the Royal Society, Vols. 158, 162, 165, and 167 (years 1868 to 1877) contain Sir Edward Sabine's great collection and isomagnetic charts (declination, dip and intensity) of observations embracing the earth's surface, the epoch being between 1840 and 1845. For modern charts showing the distribution of magnetism over the globe, the reader may consult the admirable series of charts for epoch 1885 in Berghaus' "Physical Atlas," 3d Ed., published in 1892, Art. Erdmagnetismus.

For the development of the theory of terrestrial magnetism, a knowledge of the position of the poles is of great moment; hence, direct observations in their vicinity, as far as accessible, are especially valuable, yet, notwithstanding the greater experience and less danger now in Arctic traveling than was the case half a century ago, the bold dash of Sir James Ross to the American pole on the shore of Boothia, in the spring of 1831, has not been repeated, and we are yet almost in total ignorance whether or not the pole holds a nearly fixed position or is subject to secular motion, to clear up which a magnetic expedition to this region would be highly desirable.† To accomplish this effectively science would now demand a complete magnetic survey of the region surrounding the pole.

* Reproduced in the Greenwich Astronomical Observations for 1869.

† The reader desirous of further information on this matter may consult the article "An Expedition to the Northern Magnetic Pole," a discussion before the American Geographical Society, May 2d, 1892, in "Bulletin of the Society," Vol. XXIV, No. 2, June 30th, 1892.

The late explorer, Lieutenant Schwatka, thought he had evidence that in 1879–80 the position of the pole was to be found several degrees to the westward of the place where Ross had previously located it. To locate the position of the American focus of maximum total intensity is from the nature of the case impracticable, and local irregularities of distribution would defeat any attempt.

The discussion of the variations of magnetic force, especially at the observatories supplied with automatical (by photography) registering instruments, confirmed and extended our knowledge of the intimate connection of the changes of the needle with certain solar phenomena brought about probably by an inductive action of solar (electro-magnetic) radiation, as seen in the magnetic inequalities exhibited in the daily, annual and sun-spot cycles, as well as in solar rotation, and at times in irregular disturbances. Broun was the first to derive the synodic period of the solar rotation (about 26 days) from the magnetic observations at Makerstoun (1844–45); he was followed by Hornstein and other investigators who extended the research to the records of the several self-registering variation instruments.

Kreil, in 1841, and, independently, Broun, pointed out that a lunar irregularity could be traced in the magnetic records. Thus magnetism came to be regarded as a cosmical force influencing, probably by inductive interaction through the medium of the ether, the supposed magnetic condition of the larger as well as the subordinate bodies revolving about the sun. The analysis of the stream lines seen in the structure of the solar corona during eclipses led Professor Bigelow to the conclusion* that coronal poles do not lie at the extremity of its axis of rotation, and supposing a solar inductive action on terrestrial magnetism, the synodic revolution of the sun, about 26 days as deduced from the observed inequalities, might be accounted for by the periodic alternate presentation of the northern and southern solar pole towards the earth. Recently† A. Ricco has shown that the passage of large sun spots over the central meridian of the sun is generally followed by a terrestrial magnetic disturbance, the retardation being about 45½ hours.

We may now briefly refer to the sources whence our magnetic mate-

* "American Journal of Science," Vol. XLII, July, 1891. See, also, his notes on a new method for the discussion of magnetic observations, U. S. Dept. of Agriculture; Weather Bureau Bulletin, No. 2, July, 1892.

† "Astronomy and Astro-Physics," January, 1893.

rial for North America is chiefly derived. The earlier records are necessarily from navigators and are restricted to declinations (variation of compass); it was only at a later period, when it became customary among maritime nations to send out exploring expeditions, that the elements of dip and intensity were added to the record. We have but space to mention a few leading names. For the East Coast, the Gulf and West Indies: Hudson in his third voyage, 1609, skirted the Atlantic coast from Maine to North Carolina, Champlain (1604–12) coasted between Nova Scotia and Cape Cod, and, later, Bayfield, Orlebar, Barnett, Home, Milne and many others. For the Pacific Coast and Alaska: Drake (1579 on the coast of New Albion, now California), Bering (1727–29 in Alaskan waters), Cook (1768–80), La Perouse, Malaspina, Vancouver (1790–95), Kotzebue, Beechy, Lütke, Erman (1828–30), Belcher, Wilkes (1841), Maguire (1852–54) and Lieutenant P. H. Ray (1881–83), both at Point Barrow.

For the Arctic Region of North America we meet with Baffin (1616), James and John Ross (1818), Parry (1820–25), Franklin, Dr. Kane (1853–55), Dr. Hayes (1860–61), Hall (1871), Nares and Markham (1875–76), Lieutenant A. W. Greely (1881–84) and other explorers.

There were five stations located in North America by the representatives of several nations taking part in the International Polar Research of 1882–83; they were Fort Conger, Arctic America, in charge of Lieutenant Greely; Fort Uglaamie, Point Barrow, Alaska, in charge of Lieutenant Ray (United States); Kingua Fjord, Labrador (German); Godthaale, Greenland (Danish), and Fort Rae, British Possessions (English). At these stations an enormous number of observations was accumulated and still awaits further discussion.

Of land expeditions, that of Lieutenant Lefroy in Canada and the Hudson Bay Company's Territory was, perhaps, the most extensive.* Thomas Simpson and Lieutenant Younghusband also made many observations in the British Possessions. In the United States we have to mention the expedition of Major Long, U. S. A. (1819), the exploration tours of Prof. Nicollet (1832–36), Dr. Locke (1838–43), Prof. Loomis (1838–41)†, Dr. Bache, Friesach (1856–57), and other names. The observations made by army officers in charge of the geographical surveys west of the 100th meridian are very numerous, as are also the observa-

* Diary of a Magnetic Survey, etc., during 1842–44. London, 1883.

† For his extensive collection of observations, see "Sill. Jour." Vol. XXXIV, 1838, and Vol. XXXIX, 1840.

tions due to the U. S. Lake Survey, the U. S. Coast and Geodetic Survey, and the surveys of the Public Lands; further, we have the results from the surveys of International boundaries as well as interstate boundaries and the publication of the State surveys in charge of geologists; as, New York (Regents' Reports), Pennsylvania, New Jersey*; perhaps the most complete magnetic survey of a State is that of Missouri, by Prof. F. E. Nipher, of Washington University, St. Louis (1878–82); also, the magnetic surveys made under the direction of the National Academy of Sciences, 1871–76, and a great variety of other sources, notably those published by societies, associations, or from records or deeds, books of travels, historical works, and lastly from surveyors in general. Respecting the manner of conducting magnetic surveys there are two ways of proceeding, first, one or more parties to accomplish the work proposed in a period of a few years; this will answer for a small area and should be repeated at intervals of, say, a quarter of a century, in order to ascertain the secular changes (as an illustration we possess three magnetic surveys of Great Britain); or, secondly, in the case of an extended territory, as, for instance, that of the United States, observations made at any time within the area are collected and reduced to any desired common epoch by means of the known secular change; this demands a close study of the secular variations to be had only from extended records at certain selected stations. This last course has been pursued by the U. S. Coast and Geodetic Survey for the purpose of supplying its charts with the declination for the epoch of publication, together with the annual change.

Returns have been made from the following magnetic observatories (with dates added when founded and discontinued) :

1. Philadelphia, Girard Coll., Pa..... 1840–45
2. Toronto, Canada..... 1840–93
3. Sitka, Japonski Island, Alaska..... 1842–67
4. Key West, Florida..... 1860–66
5. Habana, Colegio de Belen..... 1863–93
6. Madison, Wisconsin..... 1878–81
7. Mexico, Observatorio Central..... 1879–93
8. Los Angeles, California..... 1882–89
9. Washington, D. C., U. S. Naval Observatory.. 1888–93
10. San Antonio (two stations), Texas..... 1890–93

* Prof. G. H. Cook, Trenton, 1888.

The results collected by the U. S. C. and G. Survey number several thousands; of these, 1 245 declinations were discussed for secular variation (see Appendix No. 7, C. and G. Survey Report for 1888, 7th Edition), and the results at 3 237 stations will be found discussed and chartered (isogonics) with respect to distribution in Appendix No. 11, Report of 1889; the dips (about 2 000) and intensities (about 1 500) are given and discussed in Appendix No. 6, Report for 1885, accompanied by isoclinic and isodynamic charts. We shall devote the small space remaining to the exposition of some of the laws of variation and distribution of the magnetic force within our geographical limits possessing special general interest.

It is a matter of common observation that a magnet, when light and delicately suspended, is seldom or never at rest, but is continually shifting its direction. This motion consists of oscillations of various periods, of tremors and occasionally of outbursts of violent and irregular changes due to the so-called magnetic storms; the variations in intensity are of the same character.

The variations from the normal values are studied by means of differential instruments, those of the intensity demanding two, viz.: one for the horizontal component, the other for the vertical component; hence, any laws to which these are subject have their counterparts in the dip and total intensity.* In general, declinations and intensities are subject to the same periodic and non-periodic fluctuations, and hence, give rise to similar laws.

Beginning with the solar diurnal variation, its phase depends on local time and its character is but slightly affected by geographical position, except in the higher magnetic latitudes. About sunrise the north end of the horizontal needle is generally found approaching to or near its most easterly deviation from the magnetic meridian. This extreme is reached at 8 o'clock A. M. (about), the deflection being about $3\frac{1}{4}'$. The north end of the needle then turns to the westward and reaches the second or westernmost extreme a little before $1\frac{1}{2}$ o'clock P. M., the deflection being about $3\frac{1}{2}'$; hence, total diurnal range, $6\frac{3}{4}'$. The return motion is during summer slightly interrupted by a small retarding wave during the night. The normal value of the declination is reached at $10\frac{1}{2}$ o'clock A. M., very nearly, shifting

* These relations are $\delta\Theta = \sin.\Theta \cos.\Theta\left(\frac{\delta V}{V} - \frac{\delta H}{H}\right)$ and $\frac{\delta F}{F} = \cos.^2\Theta\,\frac{\delta H}{H} + \sin.^2\Theta\,\frac{\delta V}{V}$

from about 10 o'clock A. M. in summer and to 11½ o'clock A.M. in winter; the times of elongation and the hourly values are likewise subject to an annual inequality; also the range which in January is a minimum and in August a maximum, when it is double the former value. There are but few days in the year when this diurnal variation is not noticeable or it may be obscured by irregular motions, and these happen generally in the winter season.

The annual variation of the declination consists of a small inequality with a range of about one minute of arc. The diurnal variation is also subject to a long period inequality, *i. e.*, the sun-spot cycle of about 11 years, the observed diurnal range being smallest in years of minimum spot activity and largest in years of maximum activity, with a gradual change from one epoch to the other, the ratio being about 1 to 1½.

The variation depending on the solar rotation is small in the lower latitudes; thus at Los Angeles, Cal., it is but 2¼″ either way from the normal direction, but it becomes large for stations near the magnetic pole. The resulting synodic period is about 26 days.

The lunar inequalities exhibited in the declination are likewise of small amplitude, but have the peculiarity of producing, similar to tidal action, two maxima and two minima in each lunar day (about 25 solar hours), but there is no reflex action perceptible in any way related to the sun-spot cycle. The angular deflection (half range) is about 10″ varying between 5″ and 20″ at different stations. There is an annual inequality in the lunar diurnal variation, both in times of phase and in range, depending on the seasons when the sun is north and when south of the equator. Further, there is an effect on the declination which depends on the moon's place in its orbit and relative to the sun which is as yet not satisfactorily demonstrated.

The declination change due to the varying phases of the moon has also been studied, the range between full and new moon being about 9″; the change due to the variations in the moon's declination amounts to about the same value, but the parallactic inequality ranges only 2″ between apogee and perigee.*

Of great importance to the physicist, as well as to the surveyor, is the secular variation of the declination; it is probably of periodic character, but composed of many subperiods, and is, hence, of great complexity, which renders any predictions of more than a few years ahead a

* See, for instance, Appendix No. 9, Report for 1890.

rather hazardous matter. In our complete ignorance of its cause, its laws, so far as they have become apparent, could only be derived from a tentative study of the records extending over long periods or centuries. Appendix No. 7, C. & G. S., Report for 1881 (of 312 4to pp.), is specially devoted to this subject. Thus far it was found sufficient to express the law of change of the declination by a periodic function with one or more terms of periodicity, so that in the most simple case four unknown quantities had to be determined from the observations by the method of least squares, namely, the normal value and the length, parameter and phase of the wave. The periods demanded by the observations range from about 250 years to about 350 years according to geographical position, but outside our geographical limits the range is more than double the smallest given above (which holds for our Atlantic coast stations).

The amplitude of the secular swing varies from a few degrees to an, as yet, unknown amount. The motion of the phase is very curious; thus, we find the years of the last occurrence of the stationary phase of extreme minimum west elongation (or of its extreme maximum east declination), as follows :

At Halifax, N. S.	about	1714
" Eastport, Me.	"	1753
" Boston, Mass.	"	1780
" Philadelphia, Pa.	"	1802
" St. Louis, Mo.	"	1822
" Salt Lake, Utah.	"	1873
" San Francisco, Cal.	"	1893

The secular variation wave thus swept across the country from east to west, but apparently it took a century to do this. How far this movement will continue it is impossible to say.

More complicated than the variations noted above are the so-called disturbances. These have, so far, only been treated successfully by the statistical method; but their very separation from the general mass of observations is a matter of difficulty about which opinions differ. The best method yet devised, in the writer's opinion and one followed by him, is that of Lloyd and Sabine. Disturbances may occur at any time; they cannot be predicted; they are sudden, with rapid and great fluctuations, amounting sometimes to several degrees of deflection

they have been observed in connection with luminous outbreaks on the solar surface; they often take place over large parts or even over the whole surface of the globe simultaneously; they may last a few hours or may continue even for three or more days; they are often accompanied by auroral lights and by strong earth currents. When analyzed, they exhibit a solar diurnal variation, the westerly and the easterly disturbances, however, following different laws. They have also an annual variation and seem to depend largely on the sun-spot cycle; they exhibit themselves more conspicuously at certain hours of the day. But space forbids to follow up these minutiæ, and we shall have to content ourselves with the statement that laws in all respects similar to those mentioned above for the declination hold good also for the horizontal and the vertical components of the magnetic force.

The variations of intensity, however, do not obtrude themselves on the attention of the surveyor; it is only at laboratories or in special investigation (scientific research) that they must be taken into account.

If this brief outline of a branch of terrestrial physics should prove to be the means of directing renewed attention to it, the preparation of this article would be well repaid.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

638.

(Vol. XXX.—October, 1893.)

FUNDAMENTAL UNITS OF MEASURE.

By T. C. MENDENHALL, Superintendent of the U. S. Coast and Geodetic Survey.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

Engineering is the art of construction; but to limit it to this would be to restrict its meaning much within the range of the ordinary use of the word. In a broader sense, engineering includes all operations whose object is the utilization of the forces of Nature in the interests of man. It is both an art and a science, and as a science it consists for the most part of mathematics applied to physics and mechanics. It is of necessity, therefore, a measuring science, and a congress of engineers ought, in the nature of things, to be interested in anything relating to progress in metrology.

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

Fortunately the literature of this subject is neither scanty nor difficult of access. Much attention has been given to it in all parts of the world during the last half of this century, and in the United States especially numerous and valuable papers and reports upon the subject of weights and measures have appeared during this period. Indeed, it would be difficult, if not impossible, to contribute to either the historical or the controversial aspect of the question anything new and of notable value.

In spite of an extensive and widely circulated literature, however, it is very distinctly in evidence that the origin and genesis of the units of measure in customary use in the United States and among most English-speaking people, and their relation or want of relation to each other, are matters concerning which many engineers are not well informed.

Perhaps not a large majority of members of the profession in good standing would be able to answer accurately the question, What is a yard? or What is a pound? In view of this fact, no excuse need be offered for presenting a statement of facts relating to fundamental standards in this country at the present time, and for the sake of clearness this statement will be prefaced by a brief consideration of the principles involved in the evolution and selection of standards, together with a *résumé* of genealogical history, showing their origin and ancestry. Of the latter little detail will be given, as the extensive and easily accessible literature upon the subject renders it unnecessary.

The measure of a magnitude is its numerical evaluation. In a direct way this is accomplished by ascertaining how many times it contains another magnitude of the same kind or nature, which is adopted as a unit. Thus a length is selected as a unit for the measurement of length, a volume for the measurement of volume, a time for time measurement, and so on. At first it seems that this condition of sameness of the unit and the thing measured is a necessity. A little reflection will show, however, that it is open to the objection that it naturally, although perhaps not necessarily, leads to an almost indefinite multiplication of independent units. The discovery and development of inter-relations among measurable magnitudes, which has gone on from the earliest times, has tended towards a reduction in the number of units and, consequently, to a great simplification of the

whole subject of metrology. So simple and evident a device as relating the unit of volume to the unit of length has only been satisfactorily realized in comparatively modern times; and, with a single exception, it may be affirmed that units of volume now in use were originally in no way related to units of length, most of them being of accidental and now unknown origin.

That a legal bushel in the United States must contain 2150.42 cu. ins. is convincing evidence that the foot or the yard has no place in its ancestry, and although there is a plausible explanation of the fact that a gallon contains 231 cu. ins., it points only to a modified volume and not a selected one.

Many interesting illustrations of the great advantage gained by neglecting the principle that "like measures like" might be given, and one or two will, perhaps, be found instructive. In observing that property of matter known as "conductivity," either as to heat or electricity, qualitative or relative conclusions were for a long time all that was required. It was at first sufficient to say and to know that one substance conducted heat or electricity better or worse than another, but with the advance of knowledge of physics it became desirable and often necessary to give numerical expression to these relations. In such cases as this the practice has usually been to select some particular substance which possesses the property in question in a higher or in a lower degree than any other and adopt it as a standard. Thus barely a quarter of a century ago conductivities of different bodies for heat or electricity were expressed in terms of copper or silver ; a lampblack surface was the standard for radiation or absorption and in most instances the standard was arbitrarily rated at 100. The literature of science contains many examples of elaborate and otherwise valuable investigations which are rendered quite worthless by the uncertain and unscientific units of measure employed. An example of the persistent use of this principle is to be found in the still common mode of expressing the density of matter, by referring it to the density of a certain kind of matter, namely water, the numerical representation of the ratio being known as "specific gravity." It has taken some years for even scientific men to fully appreciate the objectionable features of this sort of metrology, because it has required some time to prove beyond doubt that all kinds of copper or silver do not conduct alike, nor do all samples of lampblack radiate alike ; and also that the con-

ditions under which the density of water is constant are difficult of realization.

Another factor which has been, up to a very recent time, of first importance in the selection of standards, is the tendency to seek in Nature something of constant dimensions or invariable mass which possesses that general availability essential to adoption as standard. The nomenclature of metrology bears testimony to this. In our own customary system of weights and measures, the occurrence of such units as the foot, hand, grain, ell, etc., tells of the frequent recourse to natural units. This is not alone characteristic of earlier and ruder systems, but in modern metrology we have recorded the efforts of scientific men to realize this theoretically desirable condition in the selection of the quadrant of the earth, the length of a seconds pendulum and the wave length of a particular kind of light for linear standards. The only natural standard which up to this time can be said to have satisfied the requirements is the unit of time, which is the sidereal day. This might itself be considered a derived rather than a fundamental unit, and, indeed, it is difficult to conceive of any time unit other than one based on motion. The motion of the earth is assumed to be a uniform rotatory motion, and the unit is the duration of a single revolution. Vibratory or periodic motion seems to offer many advantages as a time standard, and various forms have been suggested from time to time. It has been shown that the period of a freely suspended invariable pendulum furnishes in practice a more uniform and constant time unit than the best clocks or chronometers. All standards of this type depend on the persistence of gravity, however, and of this we cannot be assured. The prime requisites of a standard are constancy and universal availability, and as the present time unit, the sidereal day, possesses these in a high degree, it is not likely that it will soon be supplanted. It would be extremely desirable, however, if a unit of time could be devised which would survive such terrestrial or celestial disturbances as would materially alter the revolution of the earth upon its axis. Something of this kind is necessary if time observations made during the present cycle are to be available in future ages, and it is possible that the determination of the relation of the wave length of light to the generally accepted unit of length may indirectly furnish a time unit possessing this characteristic in a high degree.

The greatest advance in the science of metrology in modern times

is essentially due to Gauss, and it consists of the so-called "absolute" system of measurement. Quite as much as to the author of this ingenious system metrologists are indebted to the celebrated British Association Committee on Units, led by Lord Kelvin, and including such men as Clerk Maxwell, Foster, Stoney, Fleeming Jenkin, Siemens, Bramwell, Adams, Balfour Stewart and Everett. Evincing a freedom from national prejudices worthy of the distinguished body which they represented, this committee placed the system of Gauss upon a firm and enduring basis by deriving its fundamental units from the only system of weights and measures, which, starting from a scientific basis and constructed upon scientific principles, has ever found favor among a considerable number of people, and which has now become well-nigh universal.

The tendency of the absolute system is towards simplicity through a reduction of the number of fundamental units to a minimum, while at the same time it affords every facility for the multiplication of derived units to meet the demands of convenience in practice. But however complex and numerous these derived units may be, they all grow out of the same elements, and are, therefore, easy of comparison and interchange.

It is not too much to say, and it is important that it should be said, that the beauty, simplicity and convenience of this system are not yet fully understood and appreciated by many engineers who might be greatly benefited by its use. As a single illustration, reference may be made to the still very general use of the foot-pound as a unit of work and energy. Let no one imagine that the objection to this unit lies in the fact that units of the metric system are not used, for kilogram-meter, which is also very common, is equally objectionable. The difficulty rests in the introduction of a variable, and in this instance unnecessary, magnitude; namely, the force of gravitation. If the fundamental units, foot, pound and second, be used, we have a unit of work sometimes called the "foot-poundal," and if the centimeter, gram, second, system be used, we have the well-known "erg." These units are vastly more convenient in practical use than their gravitation relatives, besides being invariable, whenever and wherever the units of length, mass and time are invariable.

Modern scientific metrology may be said to rest upon a few simple principles which may be summarized as follows :

The number of independent fundamental units should be minimum. In general not more than three are required.

They should be such as admit of a ready and accurate comparison with other magnitudes of the same kind. Units of length, mass and time satisfy this requirement better than any other that can be selected.

They should be capable of use for such comparisons at places and times widely separated; hence they ought to be comparatively easy of reproduction and transportation and, as far as human ingenuity can secure, invariable in their magnitude. Units of length, mass and time satisfy these conditions better than any other.

They should also be so related to each other, as far as such relation is possible, and the multiples and submultiples should be so related to them that units of every possible dimension and character for convenience in the measurement of all measurable things may be derived from them in the simplest manner, and thus be capable of the easiest reduction and interchange. The units of length, mass and time, as represented by the centimeter, the gram and the second, fulfill these requirements almost as perfectly as possible. The second falls short of the others because its multiples are not decimally derived, but its use is and has long been so nearly universal that it is not likely to be modified in that respect in the near future. Indeed, a decimal system as applied to time is much less important than when considered in relation to length and mass.

As to the constancy of these units, an arbitrary length and an arbitrary mass are much more capable of accurate reproduction than any natural units of which we now know. When reproduced in considerable numbers and of the best known material, and when widely distributed throughout the civilized world as they now are, under the direction of the International Bureau of Weights and Measures, anything like destruction or loss of the standards must be regarded as well-nigh impossible. Copied in materials of various kinds and preserved under conditions widely varying, it is hardly likely that any secular change in the standards can escape detection, and the accurate determination of the meter in light waves now in progress will afford a valuable check on the constancy of the standard of length.

It thus appears that the metric system with its derived units is to-day by far the most perfect system of metrology ever used by man, and that it lacks little of theoretical perfection. It can hardly be

denied that in one or two matters of minor importance it is susceptible of improvement, but it possesses the inestimable and unapproachable advantage of being actually in use by the great majority of civilized nations. Among the innumerable metrological schemes which have made their appearance within the past 100 years, it is quite possible that some one of them possesses advantages over that based on the meter and the kilogram, and that it would be preferred if we were starting afresh with the whole question. But we are not starting afresh, and it is certainly a cause for sincere and earnest congratulation that a system which is so rapidly advancing in public favor is as nearly absolutely perfect as is this.

Let us turn now to a brief consideration of the origin and present condition of what Lord Kelvin has justly characterized as the brain-wearying, intellect-destroying system of weights and measures in use among English-speaking people.

The fundamental unit of length is the yard, and the unit of mass is the pound. In the time of Edward II it was enacted (A.D. 1324) that three barleycorns, round and dry, should make 1 in. and 12 ins. make 1 ft. The earliest actual material standard yard, of which there is reliable account, dates back to the time of Henry VII (about A.D. 1490). In the *Transactions* of the Royal Society it is recorded that in 1742 "some curious gentlemen, both of the Royal Society of London and of the Royal Academy of Sciences at Paris, thinking it might be of good use for the better comparing together the success of experiments made in England and in France, proposed some time since that accurate standards of the measures and weights of both nations, carefully examined and made to agree with other, might be laid up and preserved in the archives both of the Royal Society here and of the Royal Academy of Sciences at Paris," and determined to bring about an exchange of copies of the standards of weight and mass of the respective countries. This led to an examination of the original standards of the Exchequer and their copies. It was found that the standard yard then in use was a square rod of brass, of breadth and thickness of about ½ in. The ends were neither exactly flat nor parallel. The standard was an end-measure and a matrix was provided for it. Near each end of the yard was stamped a crowned E and it dated from about 1588. Considered as a standard, its character was very inferior, but less so than the old standard of King Henry VII, which was examined

at the same time. This is described as an "old eight-sided rod of brass, of the thickness of about ½ in., very coarsely made, and as rudely divided into 3 ft., and one of these feet into inches." This is the standard which dates from A.D. 1490, and is the earliest known material yard.

In 1758, under instruction from a committee appointed by Parliament, John Bird constructed copies of the then existing standards (Elizabethan), one of which, a line measure, was recommended for adoption as the legal standard of length. A copy of this was made by the same artist in 1760, and is known as Bird's standard of 1760, to distinguish it from his first copies made in 1758. Although the subject received much consideration during the next half century, it was not until 1824 that any action was actually taken by Parliament. It follows that up to this date the legal standard of length in Great Britain and her colonies continued to be the very imperfect standard of Elizabeth referred to above. In 1824, however, it was finally enacted that Bird's standard of 1760 should be the fundamental unit of length, and in the same act it was provided that in case of loss it should be reproduced by means of its supposed known ratio to the length of a seconds pendulum at London. In 1834 the Parliament Houses, in one of which this standard had been preserved, were destroyed by fire. It is interesting to note that the conflagration was due to the burning of the "tallies" or sticks on which accounts had been kept by means of notches, and in the use of which the Government officials had persisted for many years after it had almost become a lost art elsewhere, thus exhibiting a conservatism characteristic of the whole course of the English Government in reference to metrology and allied sciences.

The legal standard having been destroyed in this manner, it was found impracticable to reproduce it, as had been intended, by the use of a pendulum, and accordingly a new standard was prepared under the direction of Mr. Sheepshanks from a half-dozen excellent copies of the destroyed standard which were available. This was legalized by an Act of Parliament in 1855, and is the Imperial standard yard of Great Britain to-day. It is a line measure, made of bronze, the total length being 38 ins. and the cross-section 1 in. square. At the time it was prepared several copies were produced, one of which, known as Bronze No. 11, is in the United States Office of Weights and Measures at Washington.

To recur now to standards of length in the United States, it is necessary to repeat the often-published statement that although the Constitution authorizes Congress to establish a system of weights and measures, it has never exercised this authority except in the matter of legalizing the metric system in 1866. The weights and measures in use in the Colonies before the Revolution were almost entirely those of Great Britain, and they continued in use without special legalization for a long time after independence was declared. The first Superintendent of the Coast and Geodetic Survey, Mr. Hassler, requiring an accurate standard of length in the operations of that Bureau, obtained from Troughton, of London, in 1814, a brass bar about 82 ins. long, 2.5 ins. wide and $\frac{1}{2}$ in. thick. This bar was a direct descendant of the Bird standard of 1760, a number of copies of which had been made by Troughton.

It being necessary for the Executive Departments of the Government to have some standards of weight and measure properly authenticated, for the purpose of levying taxes, duties, etc., this bar, or rather one particular yard of it, from the 27th to the 63d inch, was adopted as the standard of length. It was supposed to be precisely equal to the British standard at a temperature of 62° Fahr. A direct comparison with the copies of the new Imperial yard of 1855, however, showed that it was too long at that temperature, and this fact gave rise to the idea which found its way into scientific literature that the English and American yards were different, the latter being the longer. The action taken in the Office of Weights and Measures was simply to change the temperature at which it was a standard, so as to bring it into agreement with the English yard. As a matter of fact its use as a standard was practically discontinued, and the bronze copy of the Imperial yard was accepted in its place, together with another copy of this yard made of Low Moor iron and so designated.

It will thus be seen that, as far as the Government is concerned, we have followed the English in the matter of standards of length, and their yard and ours have always been as nearly as practicable identical.

The same is essentially true in regard to the standard of mass. There is an important difference, however, in that Congress did, in 1828, legalize a standard Troy pound for purposes of coinage. This was a copy of the British Troy pound of 1758, which in 1825 became

the Imperial standard. It is preserved in the Mint at Philadelphia, and is known as the Mint pound. The standard avoirdupois pound of the Treasury Department was derived from this Troy pound. Both are very inferior in construction and unsuitable for standards. The present Imperial standard of mass of Great Britain is a platinum avoirdupois pound. It was derived from a copy of the standard referred to above, which was lost in the burning of the Parliament Houses. As the Imperial standard and our own have thus a common ancestor, it is assumed that they are the same.

Besides these units of length and mass the executive officers of the Government adopted two units of volume, the gallon which contains 231 cu. ins. and the bushel of 2 150.42 cu. ins. They are old English measures and differ very materially from the imperial gallon and bushel now in use in Great Britain.

The above statements apply to what may be known as National or United States standards only in the limited sense that they are the standards of the executive branch of the Government. The whole subject of standards, with the exeception as to the metric system already noted was, in the absence of definite action by Congress, left to the law-making authorities of the several States. In view of the great and intelligent interests in this subject exhibited by Washington, Jefferson, Adams, Gallatin and others of the early statesmen, the omission to legislate in Congress must be attributed largely to the fact of great dissatisfaction with the present system and a hesitancy to recommend any other, however perfect it might seem to be, until it had received the test of actual trial. Realizing the danger which was impending of inharmonious and unscientific legislation by the several States, Congress decided in 1836 to encourage uniformity throughout the country by the distribution among the various State Governments of complete sets of weights and measures copied from the standards adopted in the United States Office of Weights and Measures. Some States had already legalized standards differing somewhat from these, but they were soon accepted by all, thus establishing a practically uniform system throughout the country and one in agreement with that adopted by the Government. Strictly speaking, however, each State has its own standards, and they are entirely independent of, although copied from, those in use at Washington. But, as has already been explained, the latter have not themselves been regarded as fund-

amental standards, being only copies of the Imperial standards of Great Britain, in the case of the yard, or descended from the same ancestry, as in the case of the pound, and assumed to be the same. It thus appears that practically, and until a very recent period, our whole system of length and mass measurement was made to depend upon the Imperial yard and pound of Great Britain.

The most important legislation upon this subject from the founding of the Government to the present time is the Act of Congress of July 28th, 1866, legalizing the metric system of weights and measures throughout the United States. It has not been generally recognized that this system is and has been for more than a quarter of a century the only system whose use is made legal throughout the whole country by Act of Congress. Since the passage of this Act there has been a decided advance in the use of this system among all civilized nations. This remarkable movement, in which the United States Government, through annual contributions of money and diplomatic negotiations, has had a large part, leaves no room for doubt that in the comparatively near future all mankind will be in the fullest enjoyment of the great boon of a single, universal system of weights and measures, and one as nearly perfect in form and design as could well be expected.

The recognition of this fact has led to recent action on the part of the Office of Weights and Measures at Washington, which is of such importance as to justify the repetition here of the words of Bulletin No. 26, United States Coast and Geodetic Survey, April 5th, 1893, in which it was first announced.

Fundamental Standards of Length and Mass.

"While the Constitution of the United States authorizes Congress to 'fix the standard of weights and measures,' this power has never been definitely exercised, and but little legislation has been enacted upon the subject. Washington regarded the matter of sufficient importance to justify a special reference to it in his first annual message to Congress (January, 1790), and Jefferson, while Secretary of State, prepared a report at the request of the House of Representatives, in which he proposed (July, 1790) 'to reduce every branch to the decimal ratio already established for coins, and thus bring the calculation of the principal affairs of life within the arithmetic of every man who can multiply and divide.' The consideration of the subject being again urged by Washington, a committee of Congress reported in favor of Jefferson's plan, but no legislation followed. In the meantime the executive branch of the Government found it

necessary to procure standards for use in the collection of revenue and other operations in which weights and measures were required, and the Troughton 82-in. brass scale was obtained for the Coast and Geodetic Survey in 1814; a platinum kilogram and meter, by Gallatin, in 1821; and a Troy pound from London in 1827, also by Gallatin. In 1828 the latter was, by act of Congress, made the standard of mass for the Mint of the United States, and, although totally unfit for such purpose, it has since remained the standard for coinage purposes.

"In 1830 the Secretary of the Treasury was directed to cause a comparison to be made of the standards of weight and measure used at the principal custom-houses, as a result of which large discrepancies were disclosed in the weights and measures in use. The Treasury Department being obliged to execute the constitutional provision that all duties, imposts and excises shall be uniform throughout the United States, adopted the Troughton scale as the standard of length; the avoirdupois pound to be derived from the Troy pound of the Mint, as the unit of mass. At the same time the Department adopted the wine gallon of 231 cu. ins. for liquid measure and the Winchester bushel of 2150.42 cu. ins. for dry measure. In 1836 the Secretary of the Treasury was authorized to cause a complete set of all weights and measures adopted as standards by the Department for the use of custom-houses and for other purposes, to be delivered to the Governor of each State in the Union for the use of the States respectively, the object being to encourage uniformity of weights and measures throughout the Union. At this time, several States had adopted standards differing from those used in the Treasury Department, but after a time these were rejected, and, finally, nearly all the States formally adopted by act of legislature the standards which had been put in their hands by the National Government. Thus a good degree of uniformity was secured, although Congress had not adopted a standard of mass or of length other than for coinage purposes as already described.

"The next, and in many respects the most important, legislation upon the subject was the act of July 28th, 1866, making the use of the metric system lawful throughout the United States, and defining the weights and measures in common use in terms of the units of this system. This was the first *general* legislation upon the subject, and the metric system was thus the first and, thus far, the only system made generally legal throughout the country.

"In 1875 an International Metric Convention was agreed upon by seventeen governments, including the United States, at which it was undertaken to establish and maintain at common expense a permanent International Bureau of Weights and Measures, the first object of which should be the preparation of a new international standard meter and a new international standard kilogram, copies of which should be made for distribution among the contributing governments.

Since the organization of the Bureau, the United States has regularly contributed to its support, and in 1889 the copies of the new international prototypes were ready for distribution. This was effected by lot, and the United States received meters Nos. 21 and 27, and kilograms Nos. 4 and 20. The meters and kilograms are made from the same material, which is an alloy of platinum with 10% of iridium.

"On January 2d, 1890, the seals which had been placed on meter No. 27 and kilogram No. 20, at the International Bureau of Weights and Measures, near Paris, were broken in the Cabinet room of the Executive Mansion by the President of the United States, in the presence of the Secretary of State and the Secretary of the Treasury, together with a number of invited guests. They were thus adopted as the National Prototype Meter and Kilogram.

"The Troughton scale, which in the early part of the century had been tentatively adopted as a standard of length, has long been recognized as quite unsuitable for such use, owing to its faulty construction and the inferiority of its graduation. For many years, in standardizing length measures, recourse to copies of the Imperial yard of Great Britain had been necessary, and to the copies of the meter of the archives in the Office of Weights and Measures. The standard of mass originally selected was likewise unfit for use for similar reasons, and had been practically ignored.

"The recent receipt of the very accurate copies of the international metric standards, which are constructed in accord with the most advanced conceptions of modern metrology, enables comparisons to be made directly with those standards, as the equations of the national prototypes are accurately known. It has seemed, therefore, that greater stability in weights and measures, as well as much higher accuracy in their comparison, can be secured by accepting the international prototypes as the fundamental standards of length and mass. It was doubtless the intention of Congress that this should be done when the International Metric Convention was entered into in 1875; otherwise there would be nothing gained from the annual contributions to its support which the Government has constantly made. Such action will also have the great advantage of putting us in direct relation in our weights and measures with all civilized nations, most of which have adopted the metric system for exclusive use. The practical effect upon our customary weights and measures is, of course, nothing. The most careful study of the relation of the yard and the meter has failed, thus far, to show that the relation as defined by Congress in the Act of 1866 is in error. The pound as there defined, in its relation to the kilogram, differs from the Imperial pound of Great Britain by not more than 1 part in 100 000, an error, if it be so called, which utterly vanishes in comparison with the allowances in all ordinary transactions. Only the most refined scientific research will

demand a closer approximation, and in scientific work the kilogram itself is now universally used, both in this country and in England.*

"In view of these facts, and the absence of any material normal standards of customary weights and measures, the Office of Weights and Measures, with the approval of the Secretary of the Treasury, will in the future regard the international prototype meter and kilogram as fundamental standards, and the customary units, the yard and the pound, will be derived therefrom in accordance with the Act of July 28th, 1866. Indeed, this course has been practically forced upon this Office for several years, but it is considered desirable to make this formal announcement for the information of all interested in the science of metrology or in measurements of precision."

"T. C. MENDENHALL,
"*Superintendent of Standard Weights and Measures.*

"Approved:

"J. G. CARLISLE,
"*Secretary of the Treasury.*

"APRIL 5TH, 1893."

As a result of this action, our fundamental units of length and mass are now the new international prototype meter and kilogram preserved by the International Bureau of Weights and Measures, near Paris, and our metrology is in touch with that of the civilized world. This is the second great step towards complete emancipation from the "brain-wearying, intellect-destroying" system with which we have so long been burdened, and let us hope that the time is not far distant when the desire of the author of the Declaration of Independence will be realized by "bringing the calculation of the principal affairs of life within the arithmetic of every man who can multiply and divide."

None will be more benefited personally by this action, and none can aid more effectually to hasten it, than the distinguished body to which this paper is respectfully submitted. If any argument in its favor

---

*Reference to the Act of 1866 results in the establishment of the following equations:

$$1 \text{ yard} = \frac{3600}{3937} \text{ meter.}$$

$$1 \text{ pound avoirdupois} = \frac{1}{2.2046} \text{ kilo.}$$

A more precise value of the English pound avoirdupois is $\frac{1}{2.20462}$ kilo., differing from the above by about 1 part in 100 000, but the equation established by law is sufficiently accurate for all ordinary conversions.

As already stated, in work of high precision the kilogram is now all but universally used and no conversion is required.

were needed, it would be sufficient to cite the example of one department of the great subject of engineering, namely, electrical engineering, which is doubtless represented in some degree in this Congress. But, yesterday a thing unknown, its beautifully simple units of measure and their inter-relations are as wings which have enabled it to outstrip those that persist in carrying the dead weight of an unscientific and hopelessly bad system of metrology.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

639.

(Vol. XXX.—October, 1893.)

HISTORICAL NOTES UPON ANCIENT AND MODERN SURVEYING AND SURVEYING INSTRUMENTS.

By H. D. HOSKOLD, M. and C. E.*

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

It would be exceedingly difficult, if not absolutely impossible, to determine how far back the nature, relation and application of lines and angles began to be studied and employed as a useful art in the graphic representation upon maps of part, or of the whole, of the earth's surface. Reference has been made to an ancient Sanscrit manuscript, in which it is stated that the Chaldeans estimated 4 000 steps of a camel to be equal to 1 mile, and that 66.25 of such miles were equal to 1°; and further, from that data, 24 000 miles were deduced as the circumference of the earth.

* Director of the National Government Department of Mines and Geology, and Inspector-General of Mines, of the Argentine Republic.

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

Pythagoras, Thales and Anaximander have each in turn been made to share in announcing the earth to be spherical. Archimedes estimated the circumference at 300 000 stadia, and in the year 276 B. C. Eratosthenes also determined the circumference to be 252 000 stadia; but as the exact length of the ancient stadium has not been accurately determined, no verification has resulted.

In about 52 B. C. Posidonius determined from observations made upon the star Canopius at Rhodes, and also at Alexandria, that the circumference of the earth amounted to 240 000 stadia.

Letron stated that the amplitude of the arc which Posidonius used was only 5°, and Strabo gives 400 000 stadia as the length of the arc, and also deduced therefrom that the circumference of the earth amounted to 288 000 stadia.

Ptolemy estimated 180 000 stadia as the circumference of the earth, and Posch infers that he obtained this result from ancient Chaldean data. Ptolemy also stated that a Chaldean mile was equal to 7.5 stadia, and that consequently 7.5 times 24 000 made the circumference equal to 180 000 Chaldean miles.

In the year 1827 B. C. an Arabian caliph instructed his astronomer to measure an arc of the meridian, and from that operation the circumference of the earth was estimated. In 1322 A. D. Abulfeda explained the process adopted, and stated that from the northern portion of the measured arc, 56 miles were deduced as the length of 1°, but the southern arc gave 56.66 miles as the length of 1°, the latter being equal to 71 English miles.

Referring to the question of ancient maps, we have reference to a mining map of an Egyptian gold mine constructed upon papyrus about the year 1400 B. C., which document is preserved in the Turin Museum, in Italy. In the British Museum, in London, exists also an old papyrus document written 1700 years B. C., which, however, is considered to be a copy of an older one. Its contents consist of rules for the calculation of areas of circles, triangles and trapeziums.

Hero, in his book, which was written about 285 B. C., describes such problems as were then applied to land surveying. He also introduced improvements in surveying instruments.

The great astronomer and geographer, Ptolemy, is said to have possessed a circle engraved upon stone, by which, in his observations, he estimated angular measure to single minutes. If that report is

true, we have a remarkable instance of mechanical skill and early geometrical conception and precision. Doubtless, however, the circle employed must have had a very large diameter to admit of such a degree of accuracy, because the subdivision of a degree into fractions by means of a vernier scale and micrometer was then unknown.

The well-known geometrical elements of Euclid, possessing, as they do, the basis of mathematics for all time, were probably based in part upon much older propositions of a similar nature; but he must have applied his rules to practical surveying in Egypt to a very large extent, and at a comparatively early period.

The ancient surveying instruments must have been exceedingly rude in construction, and probably consisted, for the most part, of two revolving horizontal cross-bars of wood or metal, moving round a central pin or axis mounted on the top of a perpendicular staff or rod thrust into the ground; and probably the observed horizontal angles formed by the straight lines observed between the cross-bars may have been measured by a straight line divided as a scale of equal parts, either fixed, or temporarily applied, by the observer. There is no evidence on record known to the writer, showing that circles or arcs of circles were used by the ancient observers; still, there is no reason why they should not have been.

Ogilby stated in his book of roads surveyed in 1674, that the Persian monarchs commanded an exact register to be made of the distance from station to station throughout their dominions. The great Macedonian conqueror also had his "*Mensores Itinerum Alexandri.*"

The Romans, who were at one time the conquerors of a considerable portion of the then known world, measured and recorded the length of all the principal lines of roads traveled by them in their military operations and conquests, and we know from the records constituting the ancient itinerary of Antoninus that some of these surveys ended at Alexandria along the Nile, and extended to parts of Asia and to other parts of Africa; so that if they did not possess an equal, or a superior, system of surveying to that practiced by the Chaldeans and Egyptians, they were sufficiently intelligent to adopt any other system of surveying more convenient. The itinerary of Antoninus, however, contains no evidence that the horizontal angles which existed between the different lines of roads were determined.

From the time when the Romans conquered Britain, Spain, etc., the

art of surveying apparently remained in a very rude condition for a very considerable period of time. In the time of Ferrel, it was in some measure revived and improved, and he attempted to measure a degree of the meridian between Amiens and Paris in 1525. The measuring instruments which he employed in determining the distance consisted of a revolving wheel of known circumference. His astronomical observations were made by an arc of a circle or angular instrument, then called a sector, of 5 ft. in diameter. From the operations thus conducted, he estimated the length of 1° to be equal to 365 088 English feet.

The earliest English work upon surveying known to the writer is that published by Diggs in 1550, and in it he describes a surveying circle of 24 ins. in diameter, each division of which represented 2°. It was mounted with two revolving horizontal bars, with short perpendicular plain sights fitted to the end of each bar, the whole being placed for use on the top of a staff. Diggs honored this instrument with the name "theodolite."

In the old Latin book upon mining and metallurgy published by Agricola in 1556, there is a chapter upon subterraneous surveying, and there are diagrams of the rudest classes of instruments then employed. A nude surveyor is represented in the act of taking levels with a large vertical circle, having a weighted movable index or pendulum, to determine when it was perpendicular.

Snellius is said to have commenced a system of trigonometrical surveying or triangulation as early as 1615, and he employed a chain to measure the base line. His amplitude arc was only 1° 11′ 5″ from which he deduced 55 074 toises for the length of 1°, a toise being equal to 6.3946 English ft. Afterwards his work was verified, and 1° found to contain 57 033 toises.

In the year 1655 Norwood measured with a chain the distance from London to York, and he obtained 57 424 toises for the length of 1°.

In 1681 Houghton published a small work upon subterraneous surveying, and he employed a small but rudely constructed circle mounted with a magnetic needle. Voigtel also wrote in 1586 upon the same subject.

It has been stated that Gascoign placed spider lines in the focus of a telescope attached to his surveying instruments as early as 1640, but this is somewhat doubtful. It is, however, certain that Picard applied

the telescope to angle-measuring instruments, and also placed spider lines to indicate the focus of the telescope in the year 1669. About the same time, he also measured a base line 7 miles in length, and employed an angular instrument of the sector class of 10 ft. radius with a telescope attached, and from his arc of 1° 22′ 58″, he estimated the length of 1° to be 57 060 toises.

The great mathematician and philosopher, Newton, had previously announced to the world his grand principle of universal gravitation, and he determined the form of the earth to be that of an oblate spheroid, and that consequently the equatorial axis of the earth was greater than the polar. The geodetic operations performed by Picard seem to have enabled him to establish his principles.

Between the years 1684 and 1718, the original triangulations of Picard were extended by Cassino southward to Collioure and northwards as far as Dunkirk, in France. A base line was measured at each end, and the northern portion of his arc had an amplitude of 2° 12′ 0″, from which 56 960 toises were deducted as the length of 1°. The southern arc, of 6° 19′ 0″, gave 57 097 toises for the length of 1°. The deductions thus arrived at seemed to have convinced the French that Newton's hypothesis was erroneous and serious discussions resulted between the two opposite parties. Finally, in the year 1744, the same base lines were remeasured by Cassini de Thuri and Lacaille, and from the deductions arrived at they declared that the earth was oblate, thus confirming the theory of Newton and Huygens; still, the French scientists generally were opposed, while the English as strenuously affirmed. To settle this question definitely the French agreed to undertake, in the year 1775, two scientific expeditions, one to Peru and the other to Lapland. In the latter place the angles were measured by means of a quadrant 24 ins. in diameter, fitted with a telescope and micrometer. The astronomical observations were obtained by the use of a sector 9 ft. in length. The final results deduced from these observations gave 57 437.9 toises as the length of 1° of the meridian which cuts the polar circle. Those operations were afterwards examined resulting in the length of 1° to be 57 237 toises.

The Peruvian expedition measured two bases near Quito and Cotopaxi of 7.6 and 6.4 miles, and the quadrants used for the measurement of the horizontal angles were 24 and 26 ins. in diameter; but they were of a very inferior construction. The astronomical sectors were 8 and

12 ft. radius. The length of 1° obtained was equal to 56 753 toises, and the scientific world became convinced that those deductions confirmed Newton's theory. A general account of the geodetical operations made by the two expeditions was published in France, in 1749.

It is interesting to note that Weidler published a book, in Latin, upon surveying, in 1726; and also Beyer published his work on the same subject in 1749, followed by Oppel, in German, in 1749.

The Englishman, General Roy, commenced a trigonometrical survey in Scotland, between the years 1747 and 1755; but the results were not reliable, and this induced the general opinion that surveying circles or instruments of a superior class should be constructed and introduced before accurate geodetical results could be obtained. Various surveys were, however, conducted in Ireland about 1752 and 1755, but the instruments employed were of the class known as circumferenters, very similar to that described in the old book of Diggs.

The necessity for more accurate instruments being publicly acknowledged, the celebrated Englishman, Ramsden, undertook to investigate the question, and his scientific, mathematical, mechanical and general practical knowledge led to the ever-memorable invention of his dividing engine for the purpose of graduating or dividing astronomical and surveying circles by machinery instead of by hand, as had been the case up to that time.

It is only just to remark that Bird wrote a work upon a system of graduating instruments in 1767.

The history of the progress made by Ramsden is as interesting as it is important; but it was not until he had experienced various failures that he succeeded. The division of a circle by hand before and up to the time of the discovery of the dividing engine, introduced inequality, as may be supposed, producing serious errors in the angles observed with such instruments; but the Ramsden engine reduced such inequality to a minimum, and the mechanism which he devised has not, to the knowledge of the writer, been superseded; in fact, we may go as far as to say that the division of a circle is now performed by the delicate modern dividing engine with almost mathematical precision.

The system introduced by Ramsden was soon adopted throughout Europe, and from that period geodetical surveying may be said to be placed upon a more certain and perfect basis. When the dividing engine was completed, Ramsden constructed his great theodolite of

36 ins. in diameter, the division of the horizontal circle being subdivided by means of micrometrical microscopes to read to a single second of arc. The telescope of this instrument had a focal length of 36 ins., and was the first important instrument used upon the English trigonometrical survey. Ramsden constructed a second theodolite of the same class and dimensions, which was also employed in the English geodetical operations. It is curious to note that Ramsden's first theodolite is still preserved in England in perfect condition.

In the year 1784 an English base line was measured with a steel chain which had been prepared by Ramsden, and two independent measurements of the same base were made by this chain, differing only by 1.5 in. from each other in a total distance of 5 miles. Repeated measurements of other base lines gave a less difference.

The English trigonometrical survey was delayed by the death of General Roy, and it was not until the year 1791 that it was recommenced. The detailed account of the various scientific processes adopted, and instruments employed in carrying it out, was published by Mudge & Dalby in 1799.

The repeating surveying circles employed by the French in geodetical operations were found not to be capable of competing with Ramsden's theodolite, because the peculiar construction of the former class of instrument did not insure its entire stability.

In the East Indies, in latitude 23° 18′ 0″, a degree was measured, and found to contain 56 725 toises. Laplace and Lagrange proposed, in 1791, that a standard linear unit of measure should be adopted in France, and to be made equal to $\frac{1}{10000000}$ part of the earth's quadrant, and to be called a meter. It was also proposed that the length of this quadrant should be determined by the measurement of an arc of 9° 40′ 24″, two-thirds of which was to be north of the 45th parallel, the northern terminus being Dunkirk, France, and the southern, Barcelona, in Spain. The celebrated mathematician Delambre was in charge of the former, and Mechain of the latter. The present length of the French meter is the result of their labors.

Mudge measured various base lines in England, and continued the triangulation, completing an arc of 2° 50′ 0″, from which he deduced the length of 1° in latitude 53° to be equal to 57 017 toises, and in latitude 51° it was determined to be 57 108 toises.

Biat and Arago commenced in the year 1806 to extend the triangulation from Mount Mongo, on the coast of Valencia, to Formentera, completing an arc of 12° 22′ 13.44″, the operations extending over two years. The position of Formentera was corrected by Biat in 1825, and found to differ by 9″ from the first determination.

In 1821 it was determined that the length of 1° in 45° north latitude was equal to 56 964 toises, and in 1841, Puissant discovered another error, changing the length of 1° to 57 032 toises. It is important to note that on this occasion the principle of correction by least squares was employed for the first time. Geodetic operations were not commenced in Prussia before 1802.

The Russians made some unimportant attempts to measure a base line in 1737, but no definite result was achieved. Colonel Tenner and Struve initiated a better system in 1817, and continued it until 1827. An arc of 4.5° was measured, the angles being observed by a reflecting theodolite of the French type, 16 ins. in diameter. In ten years from 1821, Struve finished an arc of 3.5°.

There being a gap between Russia and Lapland, it was determined to measure the intervening arc, and the operations were conducted by Struve and Argelander, and completed in 1844. Finally, the measurements were continued through Norway and Sweden, the operations being conducted by Struve, Selander and Hansteen. At the end of 1855 an arc of 25° 20′ 9.29″ had been completed. The Russian party employed eight base lines, 224 principal triangles, and nine astronomical determinations. The Scandinavian party used two base lines, 33 principal triangles, employing four astronomical stations.

Struve discovered that the periodic error of observation due to the mode of repetition was very considerable, and this led him to adopt the plan of measuring the same angle upon different parts of the graduated horizontal circle. The deductions arrived at by Struve from the measurement of his great arc of 25° 20′ 9.29″, were that the length of 1° in latitude 53° 20′ 0″, amounted to 57 092 toises. In 55° 34′ 0″, it was 57 166; in 56° 32′ 0″ it amounted to 57 121; in 57° 28′ 0″, it was 57 123, and in 59° 14′ 0″ it was 57 125 toises. The small number of astronomical stations selected by the Russian and Swedish Commissioners, rendered this great arc of less value than otherwise would have been the case.

In 1818 General Maufflin carried on geodetical operations with the

object of connecting Sesburg with Dunkirk, and the arc measured amounted to 8° 21′ 18″.

Brest was connected with Strasbourg by General Bonn between 1818 and 1823. The whole of the operations were carried out at night, a light being placed in the focus of parabolic reflectors for the signals at the principal stations, and the observations for the determination of the difference of longitude were aided by powder flashes.

A trigonometrical survey was made in Hanover by Gauss, who measured an arc of 2° 57′ 0″, and he calculated the length of 1° to be 57 033 toises. He also employed a heliotrope as a signal at some of the stations. The arc of 1° 31′ 53″ measured in Denmark gave 57 092 toises as the length of 1° in latitude 54° 8′ 13″.

One of the most important and extensive trigonometrical surveys on record was that commenced by Colonel Lamton in India and continued by the celebrated Colonel Everest in 1823. The latter employed a theodolite with a divided horizontal circle of 36 ins. diameter, and he extended the triangulations from 18° 3′ 0″ to 24° 17′ 0″. He was also the inventor of that beautiful type of theodolite bearing his name. This great survey was continued from 1843 to 1861 by Sir A. Waugh, and he added 8 000 miles to the Indian chain of triangles.

General Walker succeeded him, and in 13 years completed 5 500 miles of triangulations, occupying 55 azimuth stations, and determined 89 astronomical positions. The sides of the triangles employed were from 15 to 60 miles in length, and in various cases towers of masonry work were erected 50 ft. in height. Heliotrope signals were employed at the stations during the day, and Argand lamps at night.

The amplitude of the great Indian arc amounted to 23° 49′ 23.54″, but it is believed that the local attraction of the Himalayan Mountains, etc., affected the latitude and azimuth determinations. It is, however, very curious to note that when the calculated effects of such attractive forces were applied, there still existed a slight discrepancy. The principal object of this immense national survey is the formation of an exact topographical map exhibiting the area of the Empire within fixed boundary lines.

In 1871, the celebrated optician and mathematical instrument makers, Troughton & Simms, of London, constructed a new theodolite for the Indian survey, the horizontal circle of which was 36 ins. in diameter. It was very finely divided, and the graduations of the

horizontal circle were read by six principal micrometer microscopes. The telescope also had a great focal length and very large object glass. A great number of other theodolites ranging from 15 down to 5 ins. in diameter are constantly employed upon this survey. A very careful system of spirit-level operations has also been introduced, the observations being taken by instruments constructed especially large, and in this way a considerable number of elevations have been accurately determined.

During the field season of 1891, the Behar-detachment surveying party in Bengal completed a traverse survey of 1 610 sq. miles. The No. 10 party in Bombay also completed 2 536 sq. miles of detailed survey for maps on a scale of 2 ins. to 1 mile, and also 2 100 sq. miles of triangulation in the Gujaral and Mahratta country. In Burma, No. 3 surveying party completed a cadastral survey of 1 842 sq. miles, and a traverse survey of 1 142 sq. miles in the district of Shwebo, besides a topographical survey of 106 sq. miles of the Chindwin coal fields. Various other important surveys were also instituted and are at present in continuation.

The celebrated United States Coast Survey seems to have been commenced in about 1807, but a more general system was introduced in 1831. In some of the geodetical operations Borden employed theodolites with circles of 12 ins. in diameter. In 1867, from the measurement of 6 arcs, it was found that in latitude 43° and in longitude 70° 20′ 0″, the length of 1° was determined to be 111 096 m., or 57 000.3 toises. The Pamlico-Chesapeake arc of 4° 31′ 5″ was measured in 1876, and it was found that the total error due to local deflection did not amount to more than 3.5″.

The length of 1° in latitude 37° 16′ 0″ and longitude 76° 8′ 0″ was found to be 56 999 toises.

The degree of accuracy to be attained in a trigonometrical survey depends upon the precision of measurement of the primary base line, as well as the angles, and also upon the fine mathematical processes applied to reduce and adjust such discrepancies as may have arisen during the process of observing. In no country has more attention been given to the improvement and practical application of geodetic details than in the United States, as the vast amount of valuable work of this class already effected exhibits.

In the year 1834, on the great coast survey a base line was measured

at Fire Island, on the south side of Long Island, of 5.415 miles in length, in 1844 a second base line was measured in Attleboro and Sharon, Massachusetts, and a third base near the village of Epping, near Columbia, Washington County, Maine, in 1857. The distance between the Fire Island and Massachusetts bases, taken along the axial line of triangles, was 230 miles, and that between the Massachusetts and Epping bases amounted to 295 miles. An exceedingly high degree of precision was attained in those operations, for the actual measured Fire Island base differed only by 2.75 ins. from the length as calculated from the other two bases. Doubtless, when some of the more recent geodetical operations carried on in the United States are more generally made known, their precision and great importance will be thoroughly recognized by the scientific world.

The United States geodetical surveyors do not, apparently, employ theodolites of more than 12 to 14 ins. in diameter, sizes which are preferred to the larger ones.

Some of the good points requisite in theodolites of precision for geodetical operations consist of moderate lightness of construction, rigidity of parts to resist flexure and stability of working parts, accurate divisions and also subdivisions by verniers or microscopical micrometers, minimum eccentricity and great power of telescope; this should have the line of collimation made to agree with the optical axis and center of the horizontal divided circles. On no account should there exist a dead counterpoise weight on one side of the vertical circle axis, as is sometimes the case for the object of balancing the vertical circle and tangent screw apparatus. It would be far preferable to have two vertical circles with double tangent and clamp apparatus and levels. This would be a great improvement and would offer greater advantages for correct astronomical observations and other uses than is possible when only one vertical circle is attached to the instrument. It is, however, difficult to obtain an instrument which would fulfill all these conditions. The nearest approach to perfection in the construction of instruments is to be found in those made by Troughton & Simms, of London, and the opticians of the United States.

Transit theodolites of 7.5 ins. in diameter are exceedingly useful and sufficient for most ordinary purposes. In elevated regions where the air is clear and steady, as in the mountains of the Argentine Republic, such instruments would serve to observe the angles of a secondary

series of triangles in a trigonometrical survey, especially if microscopical micrometers were to be attached. Five and 6-in. theodolites are now constructed to read to 10″ of arc by that means. For exploring parties in elevated regions, these instruments would be sufficient for pioneer work, and for taking simple astronomical observations for latitude and longitude.

It has been observed that the differences were so small between the steel standard chain, and glass tubes, when employed to measure a base line, that preference was given to the chain. Borda employed a measuring instrument constructed of platina and brass. Colby measured a base line in Ireland, nearly 8 miles in length, with an error of less than 2 ins. This apparatus consisted of two bars, one of iron and the other of brass, riveted together at the centers; these bars were compensating and performed excellent work.

In the United States, the measuring bar or instrument for determining the length of the base line consisted of two bars, one of iron and the other of brass, each less than 6 m. in length. Bessel's measuring apparatus also consisted of two bars, one of iron and the other of zinc.

Various classes of instruments for measuring base lines have been introduced with varying results; but many of these forms have been abandoned.

The slight discrepancy occurring in measuring the three sides of a series of primary triangles renders it necessary to employ the ordinary corrections, and also the well-known principle of least squares; but it is curious to note, that after the most refined mathematical principles were applied and allowances made for the effect of deflection produced by the attraction of the Himalaya Mountains, etc., in India, a slight discrepancy still remained, which apparently could not be accounted for.

The practice of the world in geodetical surveying is of the greatest utility as well as scientific interest to the newer countries, such as the South American Republics, which have the great advantage of being able to select the best instruments and modes where it may be necessary to carry out such operations; advantages, in point of fact, that did not exist in the time of the first observers.

Topographical and general surveying are of the greatest practical utility as primary elements in carrying out engineering works, but the old-fashioned mode of tracing contours naturally retarded these very expensive operations. Nothing could be better than a well-combined

system of direct theodolite surveying and spirit-level operations, but when the contour lines are too close together, and the scale of the map is small, the details become very confused and are lost in the drawing; besides, in intricate chains of mountains, such as those found parallel with, and on the eastern side of the Andes, if an engineer were once to enter some of those deep gorges for the object of contour surveying, he would scarcely ever come out.

Generally speaking, fine contour observations should be laid down upon a plan to a very large scale, and then reduced by photo-zincography, or by other means which are sometimes adopted on the Ordnance Survey of England.

The method of finding distances and altitudes by the stadia and telemeter processes, has found favor in various countries, and has had much application in the United States. This plan of determining distances is well known to engineers, and needs not to be described; suffice it to say, that it is vastly superior to the old modes for getting over a large area of ground in a given time.

Some have advocated micrometrical eye-pieces attached to the eye-end of a transit telescope having one or more horizontal wires placed in the focus; but this process has not produced good results in India, arising from the fact that the movable frame carrying the horizontal wires cannot be placed in the same vertical plane as the horizontal fixed wire in the focus of the telescope.*

No trigonometrical survey has, as yet, been commenced in the Argentine Republic; consequently, all the maps have been constructed from partial railway and other surveys, and from a determination of the latitude and longitude of places.

A new system of finding distances has been invented and practiced in India, with great success, by Colonel Tanner, Indian staff officer. As a preliminary, he says :

"Several delicate and expensive instruments were already at hand with micrometrical eye-pieces attached, by which, when used in conjunction with bars of known length, horizontal distances could be measured more or less near the truth; but such instruments were too complicated and delicate in their parts to bear the rough treatment they might receive at the hands of the native establishments to whom I had mostly to look when conducting the Himalayan survey. Besides this drawback, I have not found micrometrical instruments capable

* In the United States the three cross-bars are placed on the same frame or diaphragm.

of yielding results sufficiently accurate for the purpose of obtaining traverse distances within the prescribed error admissible for work on a large scale of 56.7 ins. to 1 mile, which was laid down for the more open parts of the country that would come under my measurements."

The Colonel then describes his process of bar-subtense, which consisted in setting up a wooden bar of 20 ft. in length at any distance horizontally, and then observing the intervening angle, and calculating the distance by a well-known simple trigonometrical rule.

Each end of the bar-subtense was mounted with a circular disc, 12 ins. in diameter, to the center of each of which the vertical wire of the theodolite telescope was directed. He states that under favorable conditions of light and atmosphere a skilled observer could obtain a 3-mile distance to within 6 ft. of error; also, that a 10-ft. horizontal bar would give good results up to 1.5 miles.

The Colonel seems to have experimented with bars down to a 2-ft. Gunther's scale, blackened at the ends with 2-in. discs, and he states that he obtained good results for a 20-chain distance. He gives the following example of calculation :

Bar = 20 ft. =	30.3 links.....	Log....	1.48144
Observed angle =	20′ 11.9″.....	Co. sec.	2.23122
Log. distance =	51.60. chains.....		3.71266

It is evident that this plan would be facilitated by forming a table of distances of, say, from 2 to 200 chains and for such angles as would be employed.

Colonel Tanner states that it is an error to suppose that a distance cannot be found correctly because the angles are very small and the relation of the sides of the triangle does not approach to that of an equilateral triangle; and he further states that the results to be obtained are superior to any other short of a trigonometrical survey.

The writer has calculated a table of distances for a series of given angles and a bar subtense of 25 m. Table No. 1 commences at 0° 10′ 0″ and proceeds to 0° 10′ 25″ and the corresponding distances are 8 594.38 and 8 250.60 m., respectively. Distances are given for each second of arc, and the difference in distance for 1″ between 0° 10′ 0″ and 0° 10′ 1″ is equal to 14.30 m., and that between the angles of 0° 10′ 24″ and 0° 10′ 25″ is 13.23 m. The increment of 0° 0′ 0.1″ in the former is 1.430 m.,

and for the latter it is 1.323m., and the increment for any other part of 1″ may be easily found.

Table No. 2 has been calculated for horizontal angles from 0° 15′ 0″ up to 0° 15′ 25″, the distance for each being 5 729.73 and 5 574.75 m., respectively. The difference in distance between the angles 0° 15′ 24″ and 0° 15′ 25″ is 6.02 m., and the increment for 0° 0′ 0.1″ is 0.602 m.

Table No. 3 commences from 0° 20′ 0″ to 0° 20′ 25″, and the difference in distance between the angles 0° 20′ 24″ and 0° 20′ 25″ is 3.21 m., and for an increment of 0° 0′ 0.1″ is 0.321 m.

We observe that the difference in the distance decreases with the increase of the angle, and for this reason when very small angles are observed, say, less than 10′, an instrument very finely graduated and mounted with a powerful telescope would be required. A 9 or 12-in. transit theodolite could be constructed to read by micrometrical microscopes to a decimal part of a second, and good results could be obtained without expending time and money in the measurement of a base line.

TABLE No. 1.

Observed Horizontal Angles.	Constant Base or Subtense.	Logarithmic Sines.	Logarithmic Secants.	Calculated Distance.	Difference in Distance.
0° 10′	25	7.4637255	2 5362745	8594.38	
0 10 1″	25	7.4644487	2.5355513	8580.80	14 30
0 10 2	25	7.4651707	2.5348293	8565.83	14 25
0 10 3	25	7.4658916	2.5341084	8551.62	14.21
0 10 4	25	7.4666112	2.5333888	8537.46	14.16
0 10 5	25	7.4673296	2.5326704	8523.35	14.11
0 10 6	25	7.4680469	2.5319531	8509.25	14.10
0 10 7	25	7.4687629	2.5312371	8495.27	13.98
0 10 8	25	7.4694778	2.5305222	8481.30	13.97
0 10 9	25	7.4701915	2.5298085	8467.37	13.93
0 10 10	25	7.4709041	2.5290959	8453.48	13.89
0 10 11	25	7.4716154	2.5283846	8439.65	13.83
0 10 12	25	7.4723257	2.5276743	8425.89	13.76
0 10 13	25	7.4730347	2.5269653	8412.14	13.75
0 10 14	25	7 4737426	2.5262574	8398.40	13.74
0 10 15	25	7.4744493	2.5255507	8384.76	13.64
0 10 16	25	7.4751549	2.5248451	8371.15	13.61
0 10 17	25	7.4758594	2 5241406	8357.58	13.57
0 10 18	25	7.4765627	2.5234373	8344 06	13.52
0 10 19	25	7.4772649	2.5227351	8330.58	13.48
0 10 20	25	7.4779650	2 5220350	8317.16	13.42
0 10 21	25	7.4786658	2.5213342	8303.75	13.41
0 10 22	25	7.4793646	2.5206354	8290.40	13 35
0 10 23	25	7.4800623	2.5199377	8277.09	13.31
0 10 24	25	7.4807588	2.5192412	8263 83	13.26
0 10 25	25	7.4814542	2.5185458	8250.60	13.23

TABLE No. 2.

Observed Horizontal Angles.	Constant Base or Subtense.	Logarithmic Sines.	Logarithmic Secants.	Calculated Distance.	Difference in Distance.
0° 15′	25	7.6398160	2.3601940	5729.73	
0 15 1″	25	7.6402983	2.3597017	5723.24	6.49
0 15 2	25	7.6407800	2.3592100	5716.76	6.48
0 15 3	25	7.6412612	2.3587388	5710.56	6.20
0 15 4	25	7.6417419	2.3582581	5704.24	6.32
0 15 5	25	7.6422221	2.3577779	5697.94	6.30
0 15 6	25	7.6427017	2.3572983	5691.65	6.29
0 15 7	25	7.6431808	2.3568192	5685.37	6.28
0 15 8	25	7.6436593	2.3563407	5679.12	6.25
0 15 9	25	7.6441373	2.3558627	5672.87	6.25
0 15 10	25	7.6446149	2.3553851	5666.63	6.24
0 15 11	25	7.6450918	2.3549082	5660.42	6.21
0 15 12	25	7 6455683	2 3544317	5654.20	6.22
0 15 13	25	7.6460442	2.3539558	5648.02	6.18
0 15 14	25	7.6465196	2.3534804	5641.84	6.18
0 15 15	25	7.6469945	2.3530055	5635.67	6.17
0 15 16	25	7.6474689	2.3525311	5629.51	6.16
0 15 17	25	7.6479428	2.3520572	5623.38	6.13
0 15 18	25	7.6484161	2.3515839	5617.25	6.13
0 15 19	25	7 6488889	2.3511111	5611.14	6.11
6 15 20	25	7.6493613	2.3506387	5605.04	6.10
0 15 21	25	7.6498331	2.3501669	5598.95	6.09
0 15 22	25	7.6503043	2.3496957	5592.88	6.07
0 15 23	25	7.6507751	2.3492249	5586.82	6.06
0 15 24	25	7.6512454	2.3487545	5580.77	6.05
0 15 25	25	7.6517151	2.3482849	5574.75	6.02

TABLE No. 3.

Observed Horizontal Angles.	Constant Base or Subtense.	Logarithmic Sines.	Logarithmic Secants.	Calculated Distance.	Difference in Distance.
0° 20′	25	7.7647537	2.2352463	4297.24	
0 20 1′°	25	7.7651154	2.2348846	4293 51	3.69
0 20 2	25	7.7654769	2.2345231	4289,98	3.57
0 20 3	25	7.7658380	2.2341620	4286 42	3.56
0 20 4	25	7.7661989	2.2338011	4282.86	3.56
0 20 5	25	7.7665594	2.2334406	4279.30	3.56
0 20 6	25	7.7669197	2.2330803	4275.76	3.54
0 20 7	25	7.7672797	2.2327203	4272.22	3.54
0 20 8	25	7 7676393	2.2323607	4268.68	3.54
0 20 9	25	7,7679987	2.2320013	4265.15	3.53
0 20 10	25	7,7683577	2.2316423	4261.63	3.52
0 20 11	25	7.7687165	2.2312835	4258.11	3.52
0 20 12	25	7.7690750	2.2309250	4254.59	3.52
0 20 13	25	7.7694332	2.2305668	4251.08	3.51
0 20 14	25	7.7697910	2.2302090	4247.58	3.50
0 20 15	25	7 7701486	2.2298514	4244.08	3.50
0 20 16	25	7.7705059	2.2294941	4240.59	3.49
0 20 17	25	7.7708629	2.2291371	4237.11	3.48
0 20 18	25	7.7712196	2.2287804	4233.63	3.48
0 20 19	25	7 7715760	2.2284240	4230.15	3.48
0 20 20	25	7.7719332	2.2280668	1226.69	3.46
0 20 21	25	7.7722880	2 2277120	4223,23	3.46
0 20 22	25	7.7726435	2.2273565	4219,78	3.45
0 20 23	25	7.7729988	2 2270012	4216.34	3.44
0 20 24	25	7.7733537	2.2265463	4213.01	3.33
0 20 25	25	7.7737084	2.2262916	4209.80	3.21

Such determination of distances could not be approached in point of accuracy by ordinary chain measurement. Naturally the bar-subtense, or, as it may be termed, constant base, should be laid horizontal; and where the ground is irregular, it would have to be elevated so as to be within view of the engineer. It should be brought in a position correctly at right angles to the line to the observer. If a telescope were to be mounted at the middle of the bar-subtense, the assistant would be able to bring it into a proper position. A little practice would conduce to the achievement of this.

It has been proposed to determine the topographical features of the country by observations taken from a captive balloon by means of photography, but there would be some difficulty in steadying a balloon over a central point, except perhaps in very calm weather. The district commanded by the camera lens would have to be determined upon the surface so as not to produce overlapping when the balloon was removed to adjacent districts.

Originally, the comparatively rough trigonometrical and other surveying operations were very laborious and tedious, frequently introducing inaccuracy in the general deductions; especially was this the case before the invention of logarithms and logarithmic sines and co-sines. If we go no farther back than 1610, the mathematical tables published up to the present time are numerous; but some of the larger and more important works, such as Gardiner, 1642; Taylor, 1792, and Vega's 10-figure logarithms, published in 1791, have become very scarce.

The greatest masterpiece of work of this class, known to the writer, is that published by General Shortridge in 1873, which contains logarithmic sines, co-sines, etc., to every second of arc. This table extends to 597 pages. We now possess, therefore, means of a very comprehensive nature, by the aid of which we can perform a greater amount of arduous labor with greater facility and certainty and under more favorable circumstances than heretofore. The practice of astronomy, geodesy, navigation and general surveying is therefore a very different thing to what it was in ancient times.

Babbage mentions that there exists in the Paris observatory manuscript tables of logarithms calculated to a large number of decimal places, and he calls it a "Treasure of Calculations."

Mining surveying is extensively practiced in the countries where the mining industry has been followed. It is not only of great impor-

tance for the object of forming under-ground plans which should accurately represent subterraneous workings, but it is frequently applied in order to determine the position, direction and length of underground galleries or tunnels which may be necessary to be constructed for the more economic extraction of the minerals. Frequently such tunnels have to be driven from two opposite points at the same time upon horizontal, slow-inclined and curved planes, and then the art of surveying is rigidly put to the test. The accuracy of the meeting points of the axial line of the tunnel will depend upon the class of instrument employed, the system adopted, and the amount of care observed in the measurement of angles and lines,

In England, Fenwick was the first to introduce a general system of mining surveying in 1801, but it depended upon the use of the magnetic needle or instrument called Miner's dial. Fenwick, however, mentions a plan of observing horizontal angles upon the graduated circle of a circumferenter with vernier, which system he called "Fast needle." This instrument was necessarily very coarse, and when it was necessary to connect the underground workings with the surface, the magnetic needle had to be employed. Several other succeeding writers advocated the use of the miner's compass which is still, unfortunately, too much employed.

The writer published a treatise upon mining surveying in 1863, introducing a specially constructed transit theodolite for observing the vertical and horizontal angles. The first line in the survey generally connected with a shaft was taken as an assumed meridian for plotting the work on a plan, all the horizontal angles being reduced to this line and plotted by one setting of a circular protractor, or, which was more preferable, by co-ordinates. A table of these was prepared and published by the writer in the work referred to; consequently, trigonometrical calculations were very much facilitated. This surveying book was employed as a class-examination book in various collegiate institutions of London, the system inculcated creating, so to speak, a new school of mining surveyors.

The plan recommended in this book for connecting the underground workings, or first line in a tunnel, to the surface, was by means of direct telescopic sight up or down a shaft, and also by suspending chains when the depth was not very great.

In March and August of 1856, the celebrated civil engineer, Arthur

Beansland, introduced a similar system which was discussed in the Institute of the North-of-England Engineers. He made the connection of an underground line to the surface, or *vice versa*, by means of direct telescopic vision down a shaft, the vertical wire in the focus being made to coincide with the underground line. For this process he employed a very powerful transit instrument. The particulars of his system were published in Vol. IV of the *Transactions* of the Society of Engineers before referred to.

This system has not become general in England, for the reason, the writer thinks, that every mining surveyor may not find it convenient to purchase an expensive transit instrument which is not useful in general surveying, and from the fact that general surveying theodolites are not constructed in such a form as to admit of their being employed for this important operation.

Some years since the writer suggested to Messrs. Troughton & Simms, of London, that the vertical axis of the horizontal plates of the theodolite should be made larger and perforated so as to admit of a direct sight of the telescope down a shaft; but the design of the instrument was lost in Paris, and consequently allowed to lapse. Recently the subject was rediscussed with the same opticians, who have actually made instruments of this class for the Cape of Good Hope Survey, the idea being that the center of the theodolite may be more accurately set up over a station.

In France and the United States, theodolites are sometimes constructed with an additional telescope, which is fixed at the end of the horizontal axis carrying the ordinary telescope. Sometimes, also, the Y's supporting the horizontal axis and telescope are bent forward so as to admit of a direct sight being made by the telescope down a shaft; but the writer believes that these modes of construction are clumsy, inconvenient and unsightly.

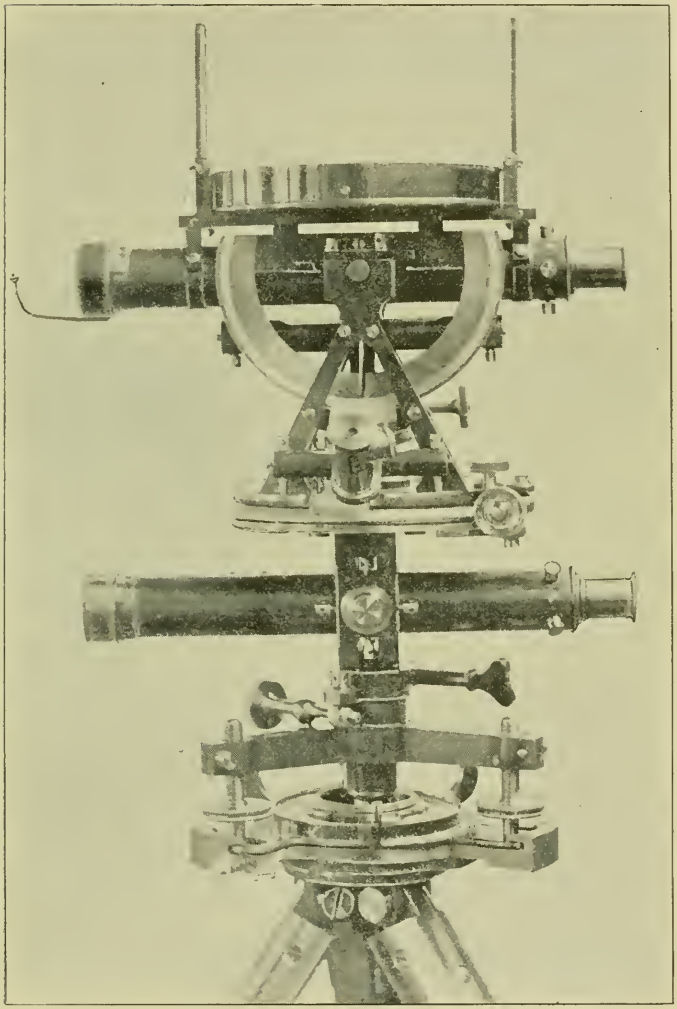
The first notice known to the writer of a similar mode of connecting the first line in an underground tunnel to the surface, or *vice versa*, is to be found at pages 249 and 250 of "Bourne's Survey," published in 1843, or 13 years before Mr. Beansland published his system. The method indicated by Bourne was employed in constructing the Box tunnel on the Great Western Railway in England. The shafts sunk upon the line of tunnel were from 300 to 400 ft. deep, and 20 ft. in diameter, and the theodolite was placed on each side of the shaft in succession in the

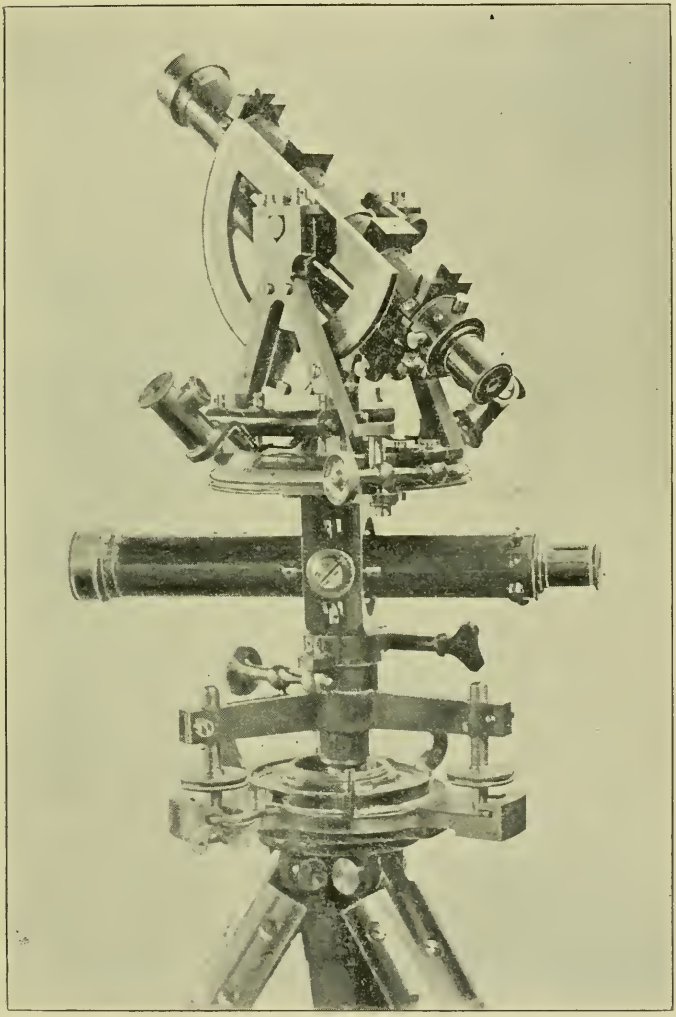
line of the tunnel; sights were then taken down the shaft and two permanent marks placed at the bottom; a line was then stretched through the marks which indicated the direction of the tunnel. Naturally this operation was repeated, and the direction checked. Browne says that when shafts are too deep, a transit instrument should be employed. This system has been much employed for railway tunnels in England.

The Severn tunnel, in England, was 2 miles in length, and shafts were put down upon it at each end or side of the river. From the top of the shafts sights were taken downward by a powerful transit instrument placed in the line of the tunnel at the surface; thus, a portion of the tunnel, 12 ft. in length, was produced at the bottom of the shaft; a wire was then stretched in the direction indicated by the vertical wire in the focus of the telescope, the wire being illuminated by electricity. It was fastened at one end and weighted at the other, and adjusted in the proper direction by lateral screws. When 100 yds. of the tunnel had been driven, its direction was again verified by observing with a transit telescope. When the tunnel met from the two ends no difference could be perceived.

The writer has executed tunnels in iron and coal mines by this means, having for a base in the bottom of the shaft, a distance of not more than 7 ft., and the error at the meeting points was inappreciable. Doubtless, excellent work has been done in the United States by this system, but the writer has no special information at hand, and is consequently unable to refer to it.

Some years since the writer also introduced a new mining and general surveying theodolite. It has two telescopes, one works in the center of the axis attached to the lower divided limb, and its optical axis coincides with the zero on that limb. The optical axis of the upper telescope coincides with the zero of the upper vernier horizontal circle. There is, therefore, no occasion to bring the zeros into coincidence. The advantages of this theodolite are various; it can be employed as a transit theodolite, and it is exceedingly useful in setting out land for colonies or locations when the lines forming the area are at right angles. Plates I and II represent a theodolite of this class of the ordinary type. Recently, however, it has received various improvements, and one of the most recently constructed types is exhibited in the British Section of the World's Columbian Exposition.





AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

640.

(Vol. XXX.—October, 1893.)

THE TREATMENT OF METALS FOR STRUCTURAL PURPOSES.

By JAMES CHRISTIE, M. Am. Soc. C. E.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

The evidence is now almost conclusive that wrought iron, as produced by the puddling process, will be largely supplanted for structural material by steel or ingot metal, as produced by fusion at high temperature. Public confidence is now fairly established in the trustworthiness of this material; the doubts of many have been gradually dissipated by the result of experience, and the manipulation of steel in all stages of its manufacture admits of such rapidity and economy of production that it will probably control future demands as the most desirable form of structural metal.

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

Hence, steel is the metal given prime consideration in this paper.

Whether blown in the Bessemer converter, or boiled in the open hearth, or whatsoever refractories may be used to facilitate desirable reactions or combinations, the general characteristics of the metal remain identical. The engineer demands results of the highest standard, without stipulating how these results may be produced; and he is only justified in specifying a certain process of manufacture, or the limitation of certain combining elements, when it is definitely known that the system specified yields more certain and uniform results than otherwise. In the open-hearth process the character of the product can be so readily controlled and predetermined, that, when metal of definite uniform composition is desired, experience favors the latter rather than the pneumatic process. On the contrary, when a large output is required of a grade readily obtained from the converter, with moderate uniformity, the Bessemer process takes precedence.

Prevalent opinion and practice favor the use of steel in structures varying between 60 000 and 70 000 lbs. tensile strength per square inch of sectional area. For special purposes, as in rivets, etc., a weaker (softer) metal is desirable; or, the known advantages of a stronger (harder) metal for compression members, etc., is an inducement to its use, the principal objection being its susceptibility to hardening influences, which in ordinary methods of production may result in sufficient hardness to render cutting processes difficult, or in extreme cases material may be in a doubtful condition for service in structures. Again, as the several grades possess a uniform coefficient of elasticity, the seeming advantage of high tensile strength is not always available, owing to necessary limitation of extension or compression and deflection. The adoption of a uniform grade for general purposes would simplify inspection, lessen the chances of the various grades being misapplied, and contribute to economy of production.

Hardening Tests.—The table gives results of a current series of hardening tests of O.-H. steel, made on silica bottoms. Three-inch ingots were hammered to a uniform section of ¾ in. square, then reheated to a bright red, and quenched in water of about 80° Fahr. The specimens thus hardened were bent with a light power-hammer to a U shape, the radius of bend being not greater than thickness of specimen. All specimens below .13 carbon bent without cracking; all above .23 carbon broke. Tensile tests for steel treated in this manner are given hereafter.

Carbon per cent.	Number bent without crack.	Number cracked.	Number breaking.	Per cent. without crack.	Per cent. cracking.	Per cent. breaking.
.13	17	1	0	94.4	5.6	0
.14	32	5	0	86.5	13.6	0
.15	30	6	2	78.9	15.8	5.3
.16	40	10	3	75.5	18.9	5.6
.17	35	18	7	58.3	30	11.7
.18	18	11	7	50	30	19.4
.19	16	5	14	45.7	14.3	40
.20	5	3	12	25	15	60
.21	4	5	6	26.7	33.3	40
.22	4	3	4	36.4	27.3	36.4
.23	1	0	7	11.5	0	87.5

It is not intended here to discuss the quality of steel as determined by the component elements, a subject which has received ample attention at the hands of competent investigators. Suffice it to say, that it is generally recognized that phosphorus and sulphur exercise an influence entirely evil, especially the former in its effects on material when in service. If the manufacturer could banish them entirely, he would do so.

Conservative specifications usually limit both these elements from .08 to .10% for O.-H. steel by acid process or one-third less for metal made on basic linings. Manganese is a necessary corrective, and is usually present to the extent of .4 to .6%, and, if ferro-silicon is used in the manufacture, silicon may be found from .10 to .15% without any known disadvantage. Carbon will vary from .15 to .25% in the structural steels under consideration.

It is desirable that the cast ingots should solidify thoroughly as porosities are apt to develop into seams and flaws, and central cavities or "piped ingots" may leave concealed fissures in finished work. To promote soundness, the fluid steel, if made in the open hearth, should remain in the furnace until ebullition subsides as far as practicable. The addition of certain reagents, notably ferro-silicon, or minute quantities of aluminum, has a decided effect in repressing the effervescence. Bottom pouring also contributes towards solidity, and the ingots should be kept in a vertical position until solid, to prevent the formation of longitudinal cavities in the fluid interior.

If before rolling, the ingots should have lost the initial heat, it is preferable to reheat slowly in a furnace of moderate temperature, as cracking is liable to ensue if cold ingots are charged into a very hot furnace.

The limit of high temperature for rolling or forging is imposed on the workmen by the necessities of the case, as excessive or prolonged overheat will result in rupture of the ingot when reduced in the mill.

In the rolling process, especially for bars of a flanged section, the manufacturer is controlled by the necessity of maintaining a proper distribution of the metals in the successive passes through the rolls; the natural flow of the heated metal should be followed by suitable shaping of the successive grooves, so that, so far as possible, all surfaces should be acted on by the roll, preventing cracking of edges, or rupture by unequal tension.

The temperature at which finished product leaves the rolls exercises some influence on its molecular structure. There is but little possibility of metal being worked at too low temperature, but if rolling ceases at too high temperature, it is claimed that a more open grain, or coarser crystallization than otherwise, is observable in fractured steel, with reduced ductility exhibited by tensile test. A similar result is sometimes found with iron where bars of large section finished very hot, when nicked with a chisel and bent, break with crystalline fracture, the fracture being fibrous if the same bars are reduced to smaller section, or if annealed. This has been repeatedly demonstrated, but it is not definitely known whether the condition described may not be due to a peculiarity of certain metals, rather than a property of all forms of malleable iron, or that the effect may be due to prolonged heating, as some experiments described hereafter at least imply a doubt on the subject. Whatever the cause, the final condition of tension bars of unusually large section should be a matter of solicitude to the inspector, and it is probable that such bars, whether of steel or iron, are benefited by final annealing.

The material should be delivered from the rolls, so as to require the least amount of cold straightening. When cold straightening is necessary, it should be done by rolling process rather than by the bending press. No injury has been found to result from roller straightening, and it is quite probable that its effect on the material is in a minor degree very similar to the well-known process of cold rolling.

Testing.—The testing of material may be either for the purpose of ascertaining its quality, or the physical effects produced by treatment subsequent to rolling. It is desirable always to select material in the

condition in which it enters the structure, and to avoid the destruction of a considerable quantity of material it is the usual custom to cut selected specimens that will, as fairly as practicable, represent material under inspection. A specimen from each melt may fairly represent the material in all the ingots cast from that heat, but a specimen from each charge of a reheating furnace would furnish little indication of the effects of reheating on the whole charge. To obtain this result it is evident that a specimen from each separately heated bloom would be necessary. It is evident that this is a matter that depends on the critical inspection desired by the purchaser, and will be a subject of mutual agreement between him and the manufacturer. Considerable confidence must be placed on the skill and honesty of manufacturers and workmen of established reputation.

Tension tests by the well-known process afford satisfactory evidence of the tensile strength and ductility of material. Drifting and bending tests are a ready and efficient means of testing ductility, that appeal to the eye and judgment, but as ordinarily practiced do not yield a result capable of being precisely recorded. Quenching heated specimens in water and bending, give evidence of the existing extent of hardening property, and the drop gives the crucial test of ability to resist shock and violence.

The various methods of testing material are so well understood, and so extensively practiced, that no radical innovation on current practice can be suggested. But if rapid testing of every piece separately rolled is desired, it may be suggested that a cropping from each piece could be mechanically punched and drifted, so that the resistance to drifting could be accurately recorded and interpreted into a reasonable measure of tensile strength, while the amount of dilation, previous to fracture, affords an approximate estimate of ductility.

The table on the next page gives the general physical properties of steel between extremes of 60 000 and 70 000 lbs. tensile strength. They are the combined averages of a series of current tests of material of approved quality, and embrace results of about 100 consecutive tests.

Shop Treatment.—It has heretofore been the usual custom to use the punch and shears for the preparation of structural iron. As steel became extensively used an impression gained that similar treatment was especially injurious to this metal, and as long as the raw material was of greater pecuniary value than iron there existed considerable disinclina-

Rolled Steel.

Tensile strength in pounds per square inch.				Elong. per cent.			Red. of Area		
Ultimate.	Elastic Limit.			In 8 ins.			Per Cent.		
	Max.	Min.	Av.	Max.	Min.	Av.	Max.	Min.	Av.
60 000	44 070	30 530	38 170	32	24	27.6	65.3	47.2	56.5
62 000	43 580	36 530	39 010	32	25	27.2	69.9	27	57.8
64 000	45 190	36 530	39 980	30.5	23.7	26.0	65.5	52.16	55.5
66 000	45 020	37 860	41 000	39.75	25.5	27.2	64.3	47.1	54.7
68 000	45 460	38 900	41 070	52.5	22.5	27.8	61 5	24	50.7
70 000	45 990	38 960	43 270	28.25	22	24.9	59.8	36.5	52.1
72 000	47 200	39 440	44 030	27	20.7	24.8	58	31.7	51.3

tion to use it unless all cut edges were planed, and holes either reamed after punching or drilled from the solid. There is no doubt that steel of higher tensile strength than is now accepted for structural purposes should not be punched or sheared, or that the softer material may contain elements prejudicial to its use however treated, but especially if punched. But extensive evidence is now on record, indicating that steel of good quality, in bars of moderate thickness, and below or not much exceeding 80 000 lbs. tensile strength, is not any more, and frequently not as much injured as wrought iron by the process of punching or shearing.

In addition to minor data on the subject, published in the technical journals, reference is made to a compilation of recent data by F. H. Lewis, C. E., in the "Proceedings of the Engineers' Club of Philadelphia," January, 1892. There is also subjoined the results of a series of experiments made within the past two years under the observation of the writer. The latter were made by skilled operators with the sole intention of getting the facts of the case.

General Effects of Shearing and Punching.—The cutting tool compresses or displaces the material immediately at the edges cut, slightly reducing the thickness, the amount of reduction being dependent upon the thickness of the material cut. Iron sheared in the direction of the fibers is usually much more roughened than when cut across the fibers. Steel usually shows a much smoother sheared edge than iron, but sometimes very minute cracks are observable across the edge of sheared steel. It is known that the effects of the disturbance are very superficial, as the removal of the visible

roughness by planing or reaming respectively for sheared and punched surfaces apparently re-establishes the original condition. The physical effects of punching and shearing as denoted by tensile test are for iron or steel:

Reduction of ductility.

Elevation of tensile strength at elastic limit.

Reduction of ultimate tensile strength.

In very thin material the superficial disturbance described is less than in thick; in fact, a degree of thinness is reached where this disturbance practically ceases. On the contrary, as thickness is increased, the injury becomes more evident.

The effects described do not invariably ensue; for unknown reasons, there are sometimes marked deviations from what seems to be a general result.

By thoroughly annealing sheared or punched steels the ductility is to a large extent restored, and the exaggerated elastic limit reduced, the change being modified by the temperature of reheating and the method of cooling. See experiments on annealing hereafter. The appearance of the fracture is appended to some of the records.

Fractured sections of steel may exhibit either a clear crystalline structure, or a dull semi-fibrous appearance, usually known as "silky," and especially observed in bars of uniform section, which, when pulled apart, stretch considerably and show considerable reduction of the fractured area, generally also breaking with a raised and depressed coning or a beveling of the broken surfaces. The character of the fracture is to some extent indicative of the method of fracture, or of how fracture is begun, rather than quality of material. Bars nicked and broken transversely show a crystalline fracture, as may also punched bars of considerable thickness when pulled to rupture, while the same punched bars reamed may exhibit a silky fracture when pulled. Illustration of this action is given hereafter. It will be observed that the disturbance of the material by the treatment, especially by shearing, was much greater in the tested specimens, than would occur ordinarily in practice, owing to the extent of disturbed surface being large in proportion to the limited dimensions of the specimen. The effects of treatment, as denoted in the experiments, are therefore an exaggerated record of what ordinarily occurs in practice.

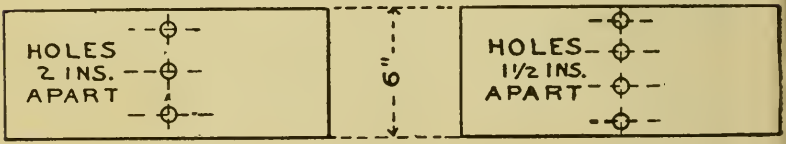
Terms are abbreviated as follows:

Ultimate tenacity in pounds per square inch of original section..... T. S.
Tensile strength at elastic limit in pounds per square inch of section..... E. L.
Stretch per cent. in 8 ins. of the fractured specimen St.
Reduction per cent. of fractured area..... . Red.

The elastic limit was recorded at the point when the scale beam dropped, sometimes given as the yield point. Stretch measured in a length of 8 ins., excepting perforated specimens, when it is measured in 2 ins., unless otherwise stated.

A series of 42 tests was made on steel plates 6 ins. wide, $\frac{1}{4}$, $\frac{5}{16}$ and $\frac{3}{8}$ in. thick, with holes $\frac{11}{16}$, $\frac{13}{16}$ and $\frac{15}{16}$ in. diameter pierced across the width as denoted below.

FIG. 1.



One-half the lot was pierced with three holes, the other half with four holes.

Tensile tests with open holes as follows, being a combined average for the series.

TABLE No. 1.

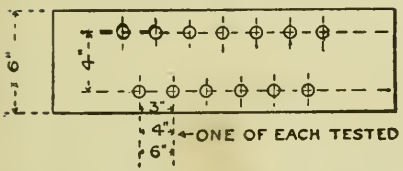
Treatment.	T. S.	E. L.	St. Per Cent.
As rolled.....	62 100	39 800	28.6 in 8 ins.
Punched.....	66 400	50 300	11.5 in 2 "
Punched and annealed,.....	66 560	42 450	15.2 in 2 "
Drilled.....	67 000	51 370	15.1 in 2 "

The abrupt reduction of sectional area on the line of the holes accounts for the elevation of tensile strength per unit of net section above that of the original bars.

TABLE No. 2.

Iron Plates Tested with Open Holes.

FIG. 2.



Original strength of the plate, an average of one test of each thickness.

T. S.	E. L.	St.	Red.
53 000	32 800	18.2	.28

Thickness.	Holes.	No. of Tests.	Total Load.	Per square inch.		St. per cent. in 8 ins.
				Max. Load.	E. Limit.	
½	Punched 15/16"	3	82 016	32 373	29 050	4.1
½	Punched 11/16" Reamed 31/32"	3	97 800	38 720	28 070	6.2
½	Punched ¾" Reamed 31/32"	3	95 766	37 850	28 550	5.1
⅝	Punched 15/16"	3	101 183	31 490	27 800	4.46
⅝	Punched 11/16" Reamed 31/32"	3	121 233	37 890	26 483	7.43
⅝	Punched ¾" Reamed 31/32"	3	123 183	38 600	27 680	6.8
¾	Punched 15/16"	3	121 266	31 746	27 963	4.76
¾	Punched 11/16" Reamed 31/32"	3	142 166	36 950	27 436	5.86
¾	Punched ¾" Reamed 31/32"	3	144 833	37 816	26 286	6.9

TABLE No. 3.

Steel Plates Tested with Open Holes. One of Each Tested.

Dimensions and Arrangement of Holes the same as in Fig. 2.

All bars rolled from one billet, the average of a test from each thickness, resulted as follows:

T. S.	E. L.	St.	Red.
64 300	39 300	28.9	57.3

Carbon, .16; Phos., .077; Mang., .51.

Thickness.	Holes.	No. of Tests.	Total Load.	Per square inch.		St. per cent. in 8 ins.
				Max. Load.	E. Limit.	
½	Punched 15/16"	3	116 283	47 563	34 040	7.46
½	Punched 11/16" Reamed 15/16"	3	131 850	54 373	32 703	10.53
½	Punched ¾" Reamed 15/16"	3	130 600	53 063	32 766	10.7
½	Drilled 15/16"	3	134 166	54 550	32 373	11
⅝	Punched 15/16"	3	143 266	46 276	32 643	7.3
⅝	Punched 11/16" Reamed 15/16"	3	170 850	54 620	31 560	10.6
⅝	Punched ¾" Reamed 15/16"	3	157 800	50 410	31 680	9.9
⅝	Drilled 15/16"	3	169 933	54 413	31 620	11.43
¾	Punched 15/16"	3	151 400	40 103	32 700	4.96

TABLE No. 4.

A series of 12 tests, made on steel bars 4½ ins. wide and $\frac{11}{16}$ in. thick, in all respects like Tables 2 and 3, except that the parallel rows of holes were 2½ ins. apart, resulted as follows:

	T. S.	E. L.	St. in 8 ins.	Red.
Bars as rolled	62 800	37 200	27.8	52
Punched $\frac{15}{16}$"	40 800	35 100	1.7	
Punched ¾" / Reamed $\frac{15}{16}$"	51 900	31 400	12.5	

In this instance every punched bar broke with a crystalline fracture, and every reamed bar with a silky fracture.

In the three foregoing cases, as the holes in specimens were not on the same line at right angles to length of bar, the effective net section was taken as the product of thickness and width of bar, less the diameter of one hole. But owing to eccentricity of strain, the specimens always broke by starting at one edge and tearing across, which accounts for the tensile strength of perforated bars being apparently so much less than the original.

TABLE No. 5.

Effect of Punching, Punching and Reaming and Drilling Steel Bars.

Bars rolled 3 ins. wide, with holes perforated through middle of bar and treated as described, gave tensile tests as follows :

Size of bar. Inches.	Treatment.	T. S.	E. L.	St.	Red.	Fracture.
3 x ⅜	As rolled	65 900	41 800	29	54	Silky.
	Punched 1"	60 800	46 100	11	13	"
	Punched ⅞" / Reamed $\frac{15}{16}$"	52 600	39 500	12	32	"
	Drilled 1"	65 500	45 000	15	29	"
3 x ⅝	As rolled	61 900	36 400	31	53	"
	Punched 1"	52 300	51 100	2.3	5.6	Crystalline.
	Punched ⅞" / Reamed $\frac{15}{16}$"	60 200	46 700	14	25	Silky.
3 x ¾	As rolled	60 300	37 300	31	49	"
	Punched 1"	46 300	44 200	7.3	3.7	Crystalline.
	Punched ⅞" / Reamed $\frac{15}{16}$"	55 000	45 700	4	27	"
3 x ⅞	As rolled	63 100	39 900	32	55	Silky.
	Punched 1"	51 000	47 200	2.3	4	Crystalline.
	Punched ⅞" / Reamed $\frac{15}{16}$"	65 900	49 400	14	20	Silky.
3¾ x ⅞ 4 tests of each.	As rolled	70 500	37 900	26.7	47	
	Drilled $\frac{15}{16}$"	73 000	39 000	10		
	Punched $\frac{11}{16}$" / Reamed $\frac{15}{16}$"	73 700	39 400	11		

The elongations were measured for solid bar in a length of 8 ins., for the perforated bars in 3 ins. except the last item of $3\frac{3}{4}$ x $\frac{7}{8}$ ins., in which elongation was measured in 2 ins. for perforated bars.

TABLE No. 6.

Effect of Shearing Edges.

Strips sheared lengthwise from the same bars of each thickness and tested in strips varying from $1\frac{1}{2}$ to 3 ins. wide.

All specimens cut from bars varying from 6 to 10 ins. wide.

Those with planed edges were previously sheared, $\frac{1}{16}$ to $\frac{1}{8}$ in. of material being removed by the planing.

Test No.	Steel Specimens. Thickness.	Treatment of Edges.	T. S.	E. L.	St.	Red.
	$\frac{5}{16}$''	Sheared	64 600	45 100	10.6	28
	$\frac{5}{16}$''	Planed	65 100	41 600	23.2	61.8
	$\frac{1}{2}$''	Sheared	68 800	43 600	16.1	28.3
	$\frac{1}{2}$''	Planed	69 000	43 000	25.7	57.8
	$\frac{11}{16}$''	Sheared	64 300	41 600	16.5	25.
	$\frac{11}{16}$''	Planed	61 600 Doubtful.	37 200	28.4	47.5
General average sheared edges			65 900	43 600	14.4	27.1
Planed edges			67 000	40 600	25.8	55.7

A series of strips $1\frac{1}{2}$ to 3 ins. wide, cut from iron bars 8 to 10 ins. wide and $\frac{3}{8}$ to $\frac{1}{2}$ in. thick, gave results as follows as a combined average of a dozen tests.

Test No.	Iron Specimen. Thickness.	Treatment of Edges.	T. S.	E. L.	St.	Red.
	$\frac{3}{8}$'' to $\frac{1}{2}$''	Sheared	42 500	33 100	5.6	18
		Planed	48 900	31 700	12.3	24.8

A bar $\frac{11}{16}$ in. thick of 7-in. flat iron, cut and tested as follows :

Test No.	Iron Specimens Width. Thickness	Treatment of Edges.	T. S.	E. L.	St.	Red.	Fracture.
	1.5'' × $\frac{11}{16}$''	Planed	47 300	Doubtful 34 200	15	21	Fibrous.
	1.6'' × $\frac{11}{16}$''	Sheared	38 300	30 400	4	13	"
	2.9'' × $\frac{11}{16}$''	Sheared	38 600	29 900	5	14	"

TABLE No. 7.

Strips of the given widths in following table were sheared from 7-in. steel bars ¼ and ½ in. thick. A portion of them had ⅛ in. planed from each edge, others were annealed by heating to redness and burying in dry sand, and some of the planed pieces, marked overheated, were heated to a bright yellow heat, forming scale, and cooled in the atmosphere. Tension tests resulted as follows :

Test No.	Steel Specimens. Width Thickness in inches.	Treatment of Edges.	T. S.	E. L.	St.	Red.	Fracture.
1	1.9 × ¼	Planed	62 200	42 800	27	60	Silky.
2	1.6 × ¼	Sheared	62 600	44 000	16	32	"
3	1.5 × ¼	Sheared and annealed	63 800	44 200	22	51	"
4	1.5 × ¼	Planed and overheated	60 700	41 100	29	60	"
5	1.9 × ½	Planed	57 300	37 200	31	59	"
6	1.6 × ½	Sheared	64 700	43 400	18	26	"
7	1.4 × ½	Sheared and annealed	61 300	41 400	26	53	"
8	1.3 × ½	Planed and overheated	61 300	37 300	30	61	"

TABLE No. 8.

Bars from same bloom, rolled 7 ins. wide, ⅜, ½ and ⅝ in. thick, all sheared lengthwise into strips and pulled as follows :

Effect of shearing steel of 80 000 lbs., T. S.; Carbon, .33; Phos., .055.

Test No.	Specimen. Width. Thickness. Inches.	Treatment of Edges.	T. S.	E. L.	St.	Red.	Fracture.
1	1½ × ⅜	Planed	79 200	45 800	22	51½	Silky.
2	1½ × ⅜	Sheared	79 700	44 500	9	23	"
3	3 × ⅜	"	79 600	43 000	14½	25½	"
4	1½ × ⅜	Sheared and annealed	83 100	50 900	19¼	31¼	"
5	1½ × ⅜	Planed and overheated	82 000	46 900	22	49¾	"
6	1½ × ½	Planed	78 500	39 600	26	52	"
7	1½ × ½	Sheared	79 400	43 400	11	15	"
8	3 × ½	"	77 700	38 800	14	21	"
9	1½ × ½	Sheared and annealed	80 800	46 900	20	27	"
10	1⅜ × ½	Planed and overheated	80 900	43 400	22	35	"
11	1½ × ⅝	Planed	78 600	41 800	24	45	"
12	1½ × ⅝	Sheared	81 100	47 200	14	14	Half crystalline.
13	3 × ⅝	"	79 600	44 400	17	19	One-fourth crystalline.
14	1½ × ⅝	Sheared and annealed	82 300	47 400	22	37	Silky.
15	1¼ × ⅝	Planed and overheated	80 800	49 700	26	53	"
16	1½ × ¾	Planed	82 200	43 500	24	50	"
17	1½ × ¾	Sheared	82 500	45 100	14	20	"
18	1½ × ¾	Sheared and annealed	83 100	43 100	20	33	"
19	1⅝ × ¾	Planed and overheated	82 500	43 300	25	50	"

It is not known definitely known if the last specimens ¾ in. thick were from the same bloom as all the others.

The planed specimens had ⅛ in. planed from each edge after shearing.

The annealing for all specimens was done by heating to redness in

a gas furnace and burying in dry sand until cool. The process seems to have been a hardening one for this grade of steel, either due to the method of cooling, or possibly the annealing temperature may have been higher than that at which the bars were originally finished. The same effect is observed in high carbon steels, unless the piece to be annealed is carefully protected from rapid cooling, or unless it is quenched at a certain critical stage of the cooling, or as usually termed water-annealed. The specimens, marked, planed and overheated, were raised to an ordinary working temperature or yellow heat, forming scale, and cooled in the atmosphere without further work. Contrary to anticipation, the action seemed to be rather beneficial than otherwise. This grade of steel, at least if low in phosphorus, shears and punches neatly, leaving a smoother surface than ordinary iron.

TABLE No. 9.

Effects of drifting holes in bars of a uniform width of 3 ins., having both edges sheared parallel with length of bar. The punched holes were perforated with a $\frac{3}{4}$-in. punch and $\frac{1}{16}$-in. die, a drilled and a punched hole being cut through each bar, and drifted until fracture was visible :

Dimensions.		Hole.	Final diameter of hole.	Fracture.
Steel 3 x $\frac{5}{8}$''	All edges sheared.	Punched $\frac{3}{4}$''	$1\frac{1}{2}$''	At edge of hole.
		Drilled $\frac{3}{4}$''	$1\frac{3}{16}$''	" "
Steel 3 x $\frac{1}{2}$''		Punched $\frac{3}{4}$''	$1\frac{5}{16}$''	" "
		Drilled $\frac{3}{4}$''	$1\frac{5}{16}$''	At edge of bar.
Steel 3 x $\frac{1}{4}$''		Punched $\frac{3}{4}$''	$1\frac{3}{8}$''	" "
		Drilled $\frac{3}{4}$''	$1\frac{3}{16}$''	At edge of hole.
Steel 3 x $\frac{1}{16}$''		Punched $\frac{3}{4}$''	$1\frac{5}{16}$''	" "
		Drilled $\frac{3}{4}$''	$1\frac{3}{4}$''	At edge of bar.
Iron 3 x $\frac{3}{8}$''		Punched $\frac{3}{4}$''	$1\frac{3}{16}$''	" "
		Drilled $\frac{3}{4}$''	$1\frac{1}{16}$''	" "

The foregoing steel was cut from the same bars tested for tensile strength on Tables Nos. 6 and 8 ; they can be compared by observing the respective thicknesses.

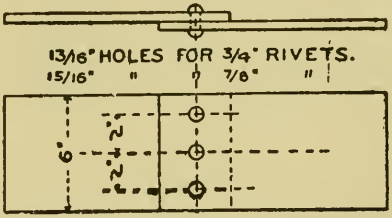
Experiments on Riveted Joints.—As the operation of riveting permanently clamps the plates together, creating a resistance to fracture of the plate independent of the force transmitted through the body of the rivet ; therefore, riveted joints can only be properly compared with each other, and not with tests of plates with open holes.

The riveted joints hereafter described were formed by lapping the ends of two plates, and fastening them with three and five rivets

respectively, as described. The object in making this imperfect joint was to obtain the distortion of the plates due to bending, when pulled in the testing machine, and thus better develop the general resisting qualities of the treated plates. As the holes were quite close together, it can be seen that the differences noted between punched and reamed joints are an exaggeration of what would occur in ordinary riveted work, or joints severed by direct tension.

TABLE No. 10.

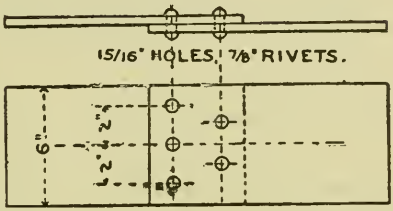
FIG. 3.



Thickness and Material.	Treatment.	Rivets.	No. of Tests.	Max. Load. Total.	Max. Load. Per square inch of net section.	Fracture.	Tests of Rolled Bars before Punching.
6 × ½" Steel	Punched ¹³/₁₆".....	3 — ¾"	2	57 200	67 100	Plate broke.....	T. S. = 62 900
	Reamed ¹³/₁₆".....	"	1	56 600	61 000	"	E. L. = 41 600
	Punched ¹⁵/₁₆".....	3 — ⅞"	2	59 700	78 700	"	St. = 31
	Reamed ¹⁵/₁₆".....	"	1	56 700	68 700	"	Red. = 68
6 × ⅝" Steel	Punched ¹³/₁₆".....	3 — ¹³/₁₆"	2	73 300	69 100	"	T. S. = 59 800
	Punched ¾"... / Reamed ¹³/₁₆"...	"	2	86 400	79 100	"	E. L. = 42 100
6 × ⅜" Steel	Punched ¹⁵/₁₆".....	3 — ⅞"	2	78 900	73 900	"	St. = 34
	Punched ¹⁵/₁₆"... / Reamed ¹⁵/₁₆"...	"	2	88 100	80 100	"	Red. = 61
6 × ½" Iron	Punched.....	"	2	71 800	46 300	"	T. S. = 54 300
	Punched and reamed ¹³/₁₆".	"	2	90 000	56 400	"	E. L. = 34 800
	Reamed ¼".....	"	2	92 000	57 700	"	St. = 17
	Drilled	"	2	94 900	59 500	"	Red. = 27
6 × ⅝" Iron	Punched.....	"	3	76 000	39 000	"	T. S. = 52 100
	Punched and reamed ¹³/₁₆".	"	2	88 500	44 000	"	E. L. = 31 300
	Reamed ¼".....	"	2	88 700	44 100	"	St. = 17
	Drilled	"	1	94 300	46 700	"	Red. = 30
6 × ¾" Iron	Punched.....	"	2	74 000	30 700	"	T. S. = 52 800
	Punched and reamed ¹³/₁₆".	"	2	96 000	38 800	Rivets sheared.	E. L = 32 100
	Reamed ¼".....	"	2	94 100	38 900	"	St. = 20
	Drilled	"	2	94 800	39 100	"	Red. = 28

TABLE No. 11.

FIG. 4.

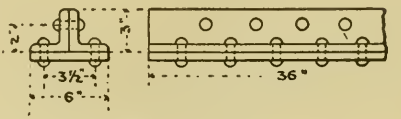


Thickness and Material.	Treatment.	Rivets.	No. of tests.	Max. Load. Total.	Max. Load. Per square inch of net section.	Fracture.	Tests of Rolled Bars before Punching.
6 × ½" Steel	Punched.....	5 — ⅞"	2	113 600	74 000	Plate broke.....	T. S. = 64 800
	Punched and reamed 3/16".	"	2	125 100	79 300	"	E. L. = 39 800
	Reamed ¼".....	"	2	126 200	80 000	"	St. = 30
	Drilled.....	"	2	119 400	75 600	"	Red. = 62
6 × ⅝" Steel	Punched.....	"	2	105 400	59 900	"	T S. = 63 800
	Punched and reamed 3/16".	"	2	151 400	76 400	"	E. L. = 38 900
	Reamed ¼".....	"	2	150 600	76 100	Rivets sheared.	St. = 28
	Drilled.....	"	2	148 700	75 200	"	Red. = 56
6 × ¾" Steel	Punched.....	"	2	89 800	38 800	Plate broke	T. S. = 64 400
	Punched and reamed 3/16".	"	2	152 000	64 000	Rivets sheared.	E. L. = 39 200
	Reamed ¼".....	"	2	154 700	65 100	"	St. = 28
	Drilled.....	"	2	156 400	65 600	"	Red. = 54

In the Tables Nos. 10 and 11 of riveted joints, some seeming inconsistencies may be observed between the quantities given for total breaking strains and the strains per unit of net section. This arises from the fact that actual thickness and diameter of holes were properly used in estimating sections, whereas the corresponding dimensions given in the table are only nominal. Several experiments have also been made on specimens, both with open and closed holes, and submitted to transverse stress. The tests recorded herewith are typical of the series.

Six ⊥ sections composed of medium grade steel plates and angles were riveted together as follows:

FIG. 5.



Angles 3 x 3 x ½ ins., plates 6 x ½ ins.
Three specimens perforated 3 ins. pitch.
Three " " 6 " "
all rivets ⅞ in. diameter.

It can be seen that the specimens were unduly weakened by the size and position of holes. These specimens were supported 32 ins. apart and pressure applied in middle, in the position shown in cut. The load was increased until fracture began.

TABLE No. 12.

Average of Specimens with—	Final Pressure.	Final Deflection.	
Holes punched $\frac{15}{16}$"	57 600 lbs.	5.78 ins.	Edge of plate and angle cracked.
Holes punched $\frac{3}{4}$" and reamed $\frac{15}{16}$"	66 700 "	6.03 "	Cracked at hole at upper edge of angle.
Holes drilled $\frac{15}{16}$"	68 600 "	6.40 "	" " " "

Tests of riveted joints have been made by subjecting specimens to rapid impact, supposed to produce strains above the elastic limit; but such tests are necessarily tedious, and no result worth describing has been attained. It is not necessary to summarize the results of the previous experiments on sheared edges or specimens with open holes, as shearing to the extent described is neither usual nor necessary, and the effects of punching should be considered on the riveted structure, or in the condition in which the material is put in service. In the riveted joints it will be observed that there is little difference resulting from the manner of perforation, when the plates are thin, the inferiority of punched as compared to reamed work becoming more marked as the plates become thicker. This is due partly to the least injury being done to the thin plates, and partly to the clamping action of the rivet heads hereafter described, a force which necessarily bears a greater ratio to the strength of a joint composed of thin plates than to one of thicker material.

For the thinnest plates there is no diminution in the strength of the punched as compared to the reamed joint; in fact, in summing up the average result, the advantage rests with the punched joints.

It may be observed here, that this is not found with imperfect or loosely driven rivets. In the ½-in. iron plates, the punched joints withstood 20% less strain than the average of reamed and drilled, while the ½-in. punched steel only fell about 6% below the punched and drilled. In the case of steel, however, the joint was made with a double row of rivets, whereas the iron was a single row, and the former resisted distortion from bending better than the latter case.

The ⅝-in. punched iron joints fell 13% below the reamed and drilled, and the ⅝-in. punched steel plates dropped 20% below the reamed and drilled. For the ¾-in. plates of both materials, no comparison can be made, as the rivets sheared in every case for reamed and drilled holes; but drilled steel plates exceeded the punched steel plates 70% without developing the full resistance of the plate. It is probable that for steel plates of good quality, and below 80 000 lbs. tensile strength, there is no material gain by drilling or reaming over punching rivet holes in plates ⅝ in. thick and less provided that rivets are solidly driven, or even in plates ½ in. thick if holes are not unusually close, and are not too near the edges. Above this thickness, there can be no doubt of the advantage of reaming punched holes. The simplicity, economy and rapidity of the punching process as compared to drilling, naturally commends it to both the manufacturer and the purchaser of structural material. With given manufacturing facilities as represented in shop area and capital invested in appliances, at least 10 times greater quantity of material can be prepared by punching than by drilling in the same time, and further improvements that may expedite production are as evident in the former as in the latter process. Estimates based on the expenditure of labor, and assuming that the cost of raw material was threefold, that of the further treatment of ordinary heavy girder work with punched holes, show approximately the following relations between costs of structural material, as differently prepared.

Material ready for shipment.	Relative cost of shop work.	Cost of material.	Total relative cost of work.
Punched work.....	1	3	4
Reamed work.....	1½	3	4½
Drilled work	3	3	6

This estimate, however, is not complete, insomuch as it does not consider the enlarged production from a given investment in plant for punching instead of drilling rivet holes. There is no evidence that material with drilled holes is superior to that with punched holes properly reamed, unless material is of excessive thickness, say, over ¾ in., or unless holes are excessively close together; and it is sometimes urged that there is no evidence of any physical inferiority of punched work as compared to reamed work, so long as strains are below the elastic limit, or until permanent deformation has begun; therefore, some engineers, rather than pay an increase of 10% for reamed work, prefer to put 10% increased material in punched work.

It is probable that the best results combined with least expenditure can be obtained by punching all holes where vital strains are not transferred by the rivets; and by reaming for important joints where strains on riveted joints are vital, or wherever perforation may reduce sections to a minimum, such as the suspension joints of floor girders, the intersections of web members with flanges in latticed girders, etc., and all holes that have to be riveted during erection of work. The reaming should be sufficient to thoroughly remove the material disturbed by punching; to accomplish this, it is best to enlarge punched holes at least ⅛-in. diameter with the reamer.

Riveting.—It is the current practice to perforate holes $\frac{1}{16}$ in. larger than the rivet diameter. For work to be reamed, it is also a usual requirement to punch the holes from ⅛ to $\frac{3}{16}$ in. less than the finished diameter, the holes being reamed to the proper size in unity, after the various parts are assembled.

It is also excellent practice, not generally insisted upon, to remove the sharp corner at both ends of the reamed holes, so that a fillet will be formed at the junction of the body and head of the finished rivets. Where holes are punched to full diameter, it is desirable that the mating holes in assembled parts should coincide so accurately that the rivets will enter freely without any forcing or drifting. In practice it is difficult to uniformly obtain such accuracy of punching; therefore some reaming is unavoidable. Hence, the best practice for punched work is to punch all holes a trifle small and ream all after assembling, the reamer being sufficiently in excess of the punch diameter to partly at least, smooth the surface of the hole. Reaming to this extent does not materially delay or increase the cost of construction, and when the

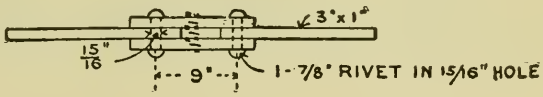
plates are not thick, say not exceeding ½ in., it is probable the results for ordinary riveted seams will compare favorably with drilled holes.

The rivets of either iron or mild steel should be heated to a bright red or yellow heat and subjected to a pressure of not less than 50 tons per square inch of sectional area.

For rivets of ordinary length, this pressure has been found sufficient to completely fill the hole. If, however, the holes and the rivets are exceptionally long, a greater pressure and a slower movement of the closing tool than is used for shorter rivets has been found advantageous in compelling the more sluggish flow of the metal throughout the longer hole. The upsetting action occurs first at the end of the rivet where the final head is being formed, the metal finally upsetting under the previously formed head of the rivet; consequently, any looseness of the rivets is more apt to occur at the end next the original head. To obviate this, it has been proposed to use straight rivet blanks, and form the heads simultaneously at both ends. It has been proved advantageous to make the rivet head elongated and containing an excess of metal, forcing this excess into the body of the rivet when closing.

Machines for closing rivets should not have a movement that terminates when the rivet is supposed to be completed, but should be capable of following the compressed material to finality. A class of machines, however, are in common use, in which the elasticity of the frame allows for inequalities, or variations in the quantity of material in the rivet. It is not usual in modern practice on structural work to use the plate-closing attachment on riveting machines, as in ordinary work no advantage has been found from its use; but if the assembled plates are exceptionally thick, or numerous, it is probable that the plate-closer does good service. It would be better in either case to allow the pressure to remain on the rivet until it had cooled sufficiently to resist any tendency of the joint to separate. Wherever hand riveting is unavoidable, the rivet should be driven by a heavy sledge, operating on the cupped tool, as by the use of light hammers the head may be completed before the body of the rivet fills the hole. Experiments for resistance of a joint with rivets closed under different pressures gave the following result.

FIG. 6.



Three specimens of 3 x 1 in. flats, secured by two $\frac{7}{8}$-in. rivets in $\frac{15}{16}$-in. holes, compressed with pressures of 70 000, 83 000 and 110 000 lbs., respectively. The pieces were then pulled in the testing machine and extensions measured in the 9 ins. between the rivets, as follows :

TABLE No. 13.

Tension applied in pounds.	Extension in inches for joint compressed with—		
	70 000 pounds pressure.	83 000 pounds pressure.	110 000 pounds pressure.
35 000	.03	.02	.025
40 000	.19	.085	.115
45 000	.25	.18	.17
50 000	.33	.26	.22
Final pressures and extensions when rivets sheared.....	51 000 — .36	55 000 — .39	56 000 — .32

The following experiment illustrates the clamping action of the rivets. 6 x $\frac{3}{8}$ ins. iron plates were perforated with six accurately reamed holes, one-half of the lot was riveted under machine pressure, as shown on the left; the remainder had straight pins driven into the holes, as shown to the right. Pulled in testing machine until plates ruptured.

FIG. 7.

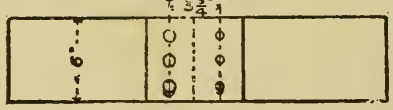


TABLE No. 14.

Diameter of holes.	Fastening.	Max. load in pounds.	Maximum load in pounds per square inch.		
			Net section of plate.	Rivet section.	Bearing area.
$\frac{11}{16}$".....	Rivets.....	81 950	52 000	36 900	64 100
	Pins.....	77 200	49 300	34 770	60 400
$\frac{13}{16}$".....	Rivets.....	75 800	53 200	24 440	50 140
	Pins.....	67 600	47 440	21 790	44 700
$\frac{15}{16}$".....	Rivets.....	73 200	57 240	17 700	42 000
	Pins.....	66 500	52 000	16 070	38 150
$1\frac{1}{16}$".....	Rivets.....	72 650	64 350	13 680	36 760
	Pins.....	57 250	50 700	10 760	28 970

For pin-connected structures, pin holes are necessarily bored and abutting surfaces faced, operations which are a subject of good workmanship and mechanical manipulation, which cannot be prejudicial to the material, and not considered in this paper.

Forging.—It is unnecessary to state that no welding should be allowed on any steel that enters into structures. The masses of greatest cross-section are used for pins. For the smaller diameters, say below 6 inches, they can be satisfactorily rolled; the larger sizes, involving insufficient reduction in the rolls, from ingots of greatest practicable section, should be forged, either under a heavy hammer, or preferably a press. The most powerful blooming mill will not close up a piped cavity in a large steel ingot that might disappear under a hammer or press of moderate capacity, and as such cavities will occasionally be found when not suspected, it is a safe practice to test excessively large pins by boring a hole lengthwise through the center; it is probable that this expense would be unnecessary on press-forged steel. Every precaution should be exercised to prevent the occurrence of such possible defects, in the event of wide tension bars of unusual thickness being required. Wherever practicable, it is desirable that the whole mass to be forged should be uniformly heated and no work performed on it after redness ceases to be visible, unless the piece is to be subsequently annealed. Pieces heated to a high temperature should be worked until sufficiently low in temperature.

It is a common belief that when metal remains heated above a certain temperature, without being compressed by manipulation, the resulting product has an open grain with inferior ductility. Numerous experiments have been made, but the demonstration is not very clear that the action is so injurious as has been claimed; that is, for the structural metal commonly known as mild or medium steel, and not containing elements to excess that might be especially detrimental to ductility under such circumstances. See some experiments in Tables Nos. 16 and 17; also, Nos. 7 and 8.

Enlarged ends on tension bars for screw-threads, eye-bars, etc., are formed by upsetting the material, and it is now abundantly demonstrated that with proper treatment and a sufficient increment of enlarged sectional area over the body of the bar, the result is entirely satisfactory. The upsetting process should be performed so that the properly heated metal is compelled to flow without folding or

lapping. If a fold occurs in upsetting, it may be cut out and the upsetting repeated; but no attempt should ever be made to weld the fissure formed by folding, as, however sound it may superficially appear, the result would be untrustworthy. The gripping tools used in upsetting should be so devised that they will not cut the surface nor impair the section of the gripped bar.

Annealing.—The object of annealing structural steel is for the purpose of securing a homogeneity of structure that is supposed to be impaired by unequal heating, or by the manipulation necessarily attendant on certain processes. The objects to be annealed should be heated throughout to a uniform temperature and uniformly cooled, and the process is not necessarily a softening one, as is inferred, when applied to high-grade steel for cutting tools, etc.

In referring to the subject of high temperatures in heated metals we are compelled, in the absence of convenient pyrometric devices, to use somewhat indefinite terms indicated by color. These terms have to be applied in the customary practical sense, allowing some latitude for different conditions and observers. For instance, a piece of metal heated visibly red in a dark shop or a gloomy atmosphere might appear to the same eye black under a brilliant light, and *vice versa.*

The proper temperature for, and results of, thorough annealing, depend on several conditions. The temperature to which the objects to be annealed should be heated depends on the existing amount of molecular disturbance that has to be neutralized. Thus, for material partially heated and forged or upset, a heat, bright red, or clean copper color, as viewed by ordinary shop light, and supposed to be about 1 200° Fahr., is found in daily practice to produce the desired results. Tests for commercial purpose are continually being made on eye-bars, annealed by heating to this extent without evidence of a higher temperature being requisite, although it is claimed by some that a lower temperature, say 1 000° Fahr., is sufficient. The temperature of 1 200° Fahr. is supposed to be about that now known as "recalescence" (probably dependent on the grade of the steel), and pieces heated not over this extent retain their form in the annealing furnace without the necessity of extremely careful and uniform support. When the physical change of material is more radical than aforesaid, as superficially occurs in shearing, punching, etc., it appears to be necessary to heat the material to a higher temperature than described, to entirely elimi-

nate the molecular disturbance. Compare tests Nos. 5, 10, 15 and 19 with 4, 9, 14 and 18, Table No. 8; also, tests 4 and 8 with 3 and 7, Table No. 7; also, see Table No. 17.

The physical effects of annealing, as indicated by tensile tests, depend on the grade of steel, or the amount of hardening elements associated with it; also, on the temperature to which the steel is raised, and the method or rate of cooling the heated material. Reference is made to some tests of annealed material on Tables Nos. 7 and 8, as well as those subjoined.

The physical effects of annealing medium-grade steel, as indicated by tensile test, are reported very differently by different observers, some claiming directly opposite results from others; and as the facts are reported by careful and impartial engineers, it is evident, when all the attendant conditions are considered, that the obtained results must vary, both in kind and degree. For the most recent statements on the subject, see those by Lewis and Dagron, Vol. XXVII, No. 4, *Transactions* Am. Soc. C. E., and Henning, Vol. XIII, Am. Soc. M. E.

The temperatures employed will vary from 1 000 to 1 500° Fahr., possibly even a wider range is used. In some cases the heated steel is withdrawn at full temperature from the furnace and allowed to cool in the atmosphere; in others, the mass is removed from the furnace, but covered under a muffle, to lessen the free radiation; or again, the charge is retained in the furnace, and the whole mass cooled with the furnace, and more slowly than by either of the other methods.

TABLE No. 15.

Four pieces cut from each of three steel bars and pulled as follows:

Test No.	Treatment of Material.		$3 \times \frac{5}{8}''$	$3 \times \frac{3}{4}''$	$3 \times \frac{7}{8}''$
1	As rolled.....	T. S.....	61 900	60 300	63 100
		E. L.....	36 400	37 300	39 900
		St.....	31	31	32
		Red	53	49	55
2	Heated red and covered in dry sand.....	T. S.....	59 200	59 400	61 000
		E. L.....	36 300	37 100	34 800
		St.....	32	28	30
		Red	55	54	48
3	Heated red and cooled in atmosphere of 70°.....	T. S.....	62 900	59 800	61 000
		E. L.....	39 500	37 700	33 700
		St	32	31	30
		Red	58	48	53
4	Heated red and quenched in water of 70°.....	T. S.....	73 400	75 000	
		E. L.....	46 400	50 600	
		St	25	23	
		Red	51	47	

All fractures silky.

TABLE No. 16.

$\frac{7}{8}$-in. round Bessemer steel bar. Cut into 17 pieces and tested as follows :

Analyses:

Carbon, .09 Mang., .41 Silicon, .005 Phos., .115 Sulphur, .082

Test No.	Treatment.	T. S.	E. L.	St.	Red.	Nicked on one side, bent to given angle before separating. Degrees.
1	As Rolled : Average of two tests.....	60 240	40 970	29	59	15
2	Heated red and covered in a box of pulverized charcoal. Average of two tests	59 860	40 300	31.5	66	12
3	Heated red and cooled in a closed box..	59 900	41 140	30.5	68.8	
4	Heated red and covered in dry sand	61 300	39 400	29	63	24
5	Heated red and cooled in atmosphere about 60°. Average of two tests... ..	61 100	41 600	31	61	48
6	Heated red and cooled in sperm oil 44° Fahr.....	65 840	46 470	27.5	54.5	
7	Heated red and cooled in water of 70° Fahr.....	73 900	47 200	19	51	
8	Heated yellow and cooled in same water	77 300	47 300	18	48	17
9	Heated yellow and cooled in air of 60° Fahr. Average of two tests	62 300	42 400	29.5	65	57
10	Yellow heat maintained for 15 minutes and cooled in air.....	63 500	42 700	24	65	30
11	Yellow heat maintained for 30 minutes and cooled in air.....	62 300	41 100	25	61	54
12	Heated yellow, forged down $\frac{1}{4}$-in. diameter, finishing at red heat. Average of two tests.....	61 600	45 100	22.6	68	

The yellow temperature was a bright, scale-forming heat as high as would be taken for forging. All fractures were silky, with the characteristic coning of the broken ends.

All specimens bent 180° without fracture after treatment.

TABLE No. 17.

Annealing Tests on Steel Bars with Sheared Edges.

A 4 × ½ in. steel flat bar was cut into 11 test strips, of which all but one had ½ in. sheared off each edge, leaving the specimens about 3 × ½ ins. Two of the latter had the edges planed and one left with sheared edges unannealed. The rest were heated and cooled as described.

Analyses :

Carbon, .34 Mang., .70 Silicon, .082 Phos., .065 Sulphur, .057

Test No.	Treatment.	T S.	E. L.	St.	Red.	Fracture.
1	Bar as rolled	79 500	40 500	27.0	48.5	S. A.
2	One-eighth in. planed off the sheared edges	79 370	40 100	25.0	43.0	"
3	One-sixteenth in. planed off the sheared edges	80 130	40 250	26.0	44.3	S. H. C.
4	Sheared edges unannealed		Record	lost		
5	Heated red and cooled in quiet atmosphere of 55° Fahr.	78 700	37 910	19.4	51.4	S. A.
6	Heated red and cooled slowly in annealing box	72 150	43 750	25.3	36.0	S. H. C.
7	Red heat retained for one-half hour; cooled in air 55° Fahr.	75 800	39 560	20.0	53.0	"
8	Red heat retained for one hour; cooled as above	74 600	41 560	16.5	51.4	S. A.
9	Red heat retained for two hours; cooled as above	74 180	38 280	14.8	51.1	{47% crystal. {53% S. C.
10	Heated yellow; cooled in quiet atmosphere of 55° Fahr.	78 560	42 550	16.3	49.1	S. A.
11	Yellow heat maintained for one-half hour; cooled as before	78 100	42 210	23.0	47.4	S. C.

S. A. — Silky-angular. S. C. = Silky-cupped. S. H.-C. = Silky-halfcupped.

No. 6 was cooled from redness in a tight-cased annealing box used for tools, but not covered under any substance; took about four hours to cool. Nos. 7 and 8 were retained at red heat in a closed muffle. Nos. 10 and 11 kept at yellow heat in the flame of an arched coke fire. Specimens, as treated, all bent 180° around a 1¼-in. pin, without cracking, except No. 1, which opened slightly on outside. Nicked and bent, all showed fine close grain, particularly Nos. 5 and 6, which were almost silky.

Specimens of steel cut from the same bar exhibit such different physical characteristics after being subjected to various temperatures or rates of cooling, that a definite result cannot be expected from annealing a miscellaneous lot of material.

We may assume that if steel was heated to a higher temperature than that at which it left the rolls, or if it was cooled more rapidly during annealing than after rolling, it would then be harder after the former process than after the latter; that is, it would probably at tensile test show higher tenacity and lower ductility. Compare tests 9 to 11 with 1 to 5 on Table No. 16, and tests 5, 10, 15 and 19 with 1, 6, 11 and 16, Table No. 8; also, tests 10 and 11 with 1 to 3 of Table No. 17. Or the contrary effects would occur, if the temperatures or rates of cooling were reversed. Thick bars of large sectional area are usually finished hotter than smaller bars, and are not liable to be made hotter during annealing ; they would therefore be more liable to be softened by the annealing than smaller sections. This would be more evident the higher the percentage of carbon in the steel.

But, on the other hand, bars reduced to small section, and finished at a low temperature, usually yield a high elastic limit, as illustrated at tests No. 12, Table No. 16, and this high elastic limit will usually be reduced by annealing. So, in a general sense, a reduction of tensile strength, and increased ductility, may be expected when steel is very slowly cooled after the annealing heat, and the contrary when it is rapidly cooled.

The best general results from annealing will probably be obtained by introducing the material into a uniformly heated oven in which the temperature is not so high as to cause a possibility of cracking by sudden and unequal changing of temperature; then gradually raising the temperature of the material until it is uniformly about 1 200° Fahr. · then withdrawing the material after the temperature is somewhat reduced and cooling under shelter of a muffle, sufficient to prevent too free and unequal cooling on the one hand, or excessively slow cooling on the other.

The foregoing statements are made with considerable reservation, as accurate knowledge of the subject has yet to be obtained.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

641.

(Vol. XXX.—October, 1893.)

A PROPOSED METHOD OF TESTING STRUCTURAL STEEL.

By ALFRED E. HUNT, M. Am. Soc. C. E.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

The methods in vogue of testing structural steel, for its quality as well as for its adaptability for the purpose required, in commercial practice at the present time are briefly as follows:

First.—Tensile tests, in which the results compared in the crucial examination are the elastic limit and tensile strength per unit of area as well as the percentage of elongation in given lengths and percentage of reduction of area.

Second.—Bending tests of the metal, both hot and cold and under varying conditions, such as quenching after being raised in temperature to a red heat, etc.

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

Third.—Drifting tests, or the enlarging of holes punched or drilled in the metal, by pressure or blows upon a hardened steel drift pin.

Fourth.—Drop tests, where specimens under given conditions of support are struck by blows of a falling weight.

Fifth.—Chemical analyses, where the amounts of the constituents imparting hardness, as well as those supposed to injure the metal by rendering it either cold or hot-short, are determined.

Some of the disadvantages of these methods of testing structural steel are: that due to the time required and expense occasioned, the tensile and chemical and in many instances the drop tests made are too few in number to discover and check a lack of homogeneity in the metal; also considerably varying results can be obtained by taking specimens from different parts of the ingot or other object to be tested, and in the case of tensile tests by varying the method of testing as to the speed, amount of impact, or the application of strains, or by pulling the specimens out of line.

A further serious disadvantage—not an inherent one, it is true, but still one which, in the writer's opinion, has grown into a grave trouble in the requirements of many specifications and in the general practice of the interpretation of specifications for structural steel of to-day—is the narrowing of the allowable limits of the results of the present system of testing to an extent that is unwise in choosing steel suitable for structural purposes, and which leads to an excessive exclusion of material. This state of affairs is not only unjust to the mills, but unsatisfactory to the shops or constructing concerns. Annoying and expensive delays are often caused by rejection of material which the mill people had confidently reckoned upon shipping at stipulated times in accordance with their promises, on account of its being just outside of the narrow limits of specifications. In many such cases there has been not the least doubt, in the mind of the inspector, the mill management, or the contractor who was to fabricate the material into finished shape, that the rejected steel would have answered equally as well for the purpose for which it had been designed as much of that which had been accepted and which had given test results within the specifications.

This state of affairs acts, in the long run, very prejudicially to the interests of the engineer who draws the specification, and sometimes actually to the quality of the metal entering into his structure; for it tends to lower the ideas of the subordinate officials and "practical

men" at both the mills and shops as to the utility of specifications and of the necessity of honestly complying with them, as well as to an idea that the engineer, the author of the specifications, does not know what material he actually wants. Added to this, not only does the rejected material cost money, but the losses resulting from delays caused by rejections are, in the long run, figured into the general cost of doing work, and are added to the estimates of costs on which bids for future orders are based. Hence the specifications under which many jobs of work are let, such as some general railroad specifications, often cause considerably higher prices to be charged and paid for, as a result of the narrow limits of the specifications, as determined by previous experience with them.

Again, bending, drifting and similar tests of ductility do not give results, in numerical quantities, convenient for reference, comparison and record, or for exact stipulation of requirements in specifications.

The method of testing which the writer proposes consists in punching or otherwise shearing, cutting or drifting pieces of a given thickness of metal, and comparing the "force" required in this work with that required to treat standard pieces in a similar manner. The comparison can be similarly made with the "work" done, or factors of the work exerted therein at different stages of the punching, cutting or drifting operations, with results obtained in treating standard pieces in a similar manner.

The term "work" is not necessarily used in its strictly scientific definition, but is used to denote the product or results of two or all of the following three factors, of which one is the force necessary to punch a hole of given size, cut a given indenture, or drift a given hole to enlarge it a given amount; the second is the space through which the force moves its point of application during the act of punching, cutting or drifting, and the third is the time during which the force acts in the cutting, punching or drifting. In the practice of the method also, a combination of the first of these factors (the force) with the second or third, is often used; and, in fact, it is a combination of the force with the space through which the force moves its point of application that the author has, so far as his experience has gone, proven to be the best and most accurate way of using it.

The most convenient way to measure these factors of the "work" done, the writer has found to be by graphical representations upon

plotting paper, where the ordinates represent the force in pounds and the abscissæ represent the increments of space through which the force moves its point of application in punching, cutting or drifting; or by graphical cards in which the ordinates represent the force in pounds and the abscissæ the increments of time during which the force is acting. Of course, in both of these species of curves, the indications of the abscissæ and ordinates may be reversed, so that the abscissæ will indicate the force in pounds and the other factor of the work shall be indicated by the ordinates.

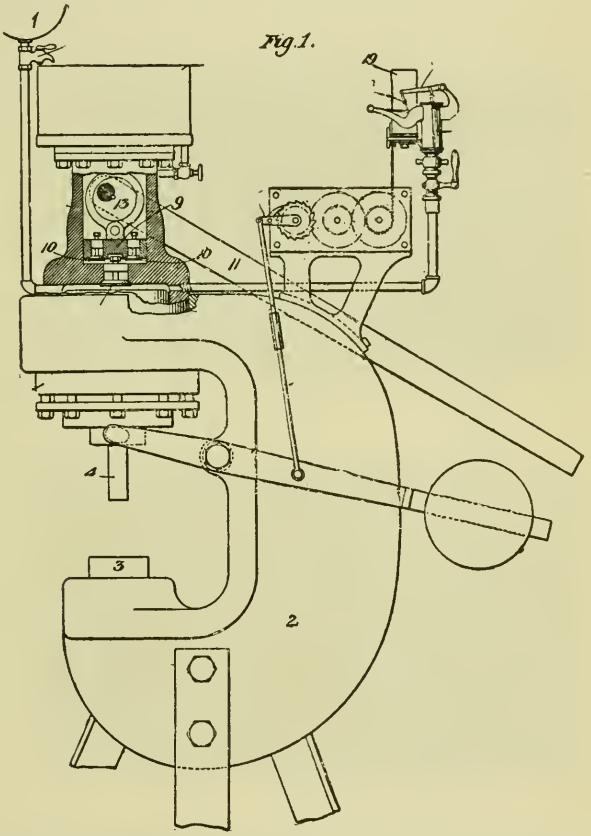
In order to produce these graphical curves, a mechanism is employed in which a sheet of ordinary section paper is placed upon a movable tablet or cylinder actuated by clockwork or other means of imparting a regular motion thereto, in the case of time being a factor of the work measured; or by some connection with the tool or some part of the mechanism which moves with the tool, in the case of the space through which the force acts being the other factor of the work employed. The pencil or stylus draws the curve, and bears upon the paper which is actuated either by a spring gauge or a measuring device constructed to connect with the punching, cutting or drifting mechanism, so that the pencil or stylus is moved in a direction transverse to the motion of the paper and a distance proportional to the pressure exerted in acting on the metal. Such methods of recording are familiar to engineers and need no further description.

In punching, cutting or drifting by this method of testing, specimens of hard steel, *i. e.*, steel deficient in ductility, will yield very slight indications of being acted upon by the working tool, up to a much higher pressure than that required for such indication in treating specimens of softer or more ductile steel.

If curves be plotted or drawn in which, as above described, the ordinates indicate the force applied and the abscissæ the amount of cutting or the time of operation of the tool, the tool will be seen to have produced much less identation in the metal under the lower loads with hard than with soft steel, and, in the case of the hard metal, the curve will much more closely approximate the perpendicular or ordinate than with the soft metal.

Where the mechanism is so arranged as to plot a closed diagram, with the ordinates indicating the force applied and the abscissæ indicating the space through which the force is applied or the time during which

it acts, the upright lines of such diagrams in the case of the less ductile steel will be much more nearly perpendicular and parallel to each other than in diagrams obtained in testing softer specimens; they will also be higher because of the greater force required to act thus upon the metal. In a general way, also, the areas enclosed by the curves of such hard pieces of steel will be less than with soft specimens.



The reason for this is, that with softer and more ductile steel the indenting work of the tool is more uniform, beginning with the exertion of a lower force, and increasing gradually as the force applied increases; but with harder and less ductile steel, comparatively little action is produced by the tool until the pressure reaches a high point, when, with much greater abruptness, the punching or cutting tool pierces the metal.

By reference to Fig. 1, an apparatus is shown for the general pur-

pose of carrying on this method of testing. The apparatus consists of an hydraulic punch, having a main frame, marked "2"; a die or anvil for the punch, marked "3," on which the specimen to be punched is placed; the punch, marked "4," which is at the end of the hydraulic piston, marked "9"; the valves of which are marked "10"; the lever actuating the punch is marked "11"; the mercury gauge is shown only in quarter outline, and is marked "1." The connection with the indi-

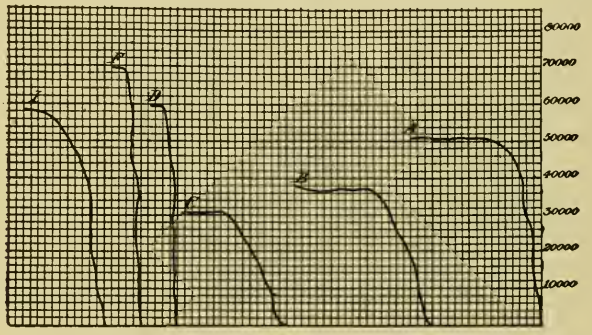


Fig. 2.

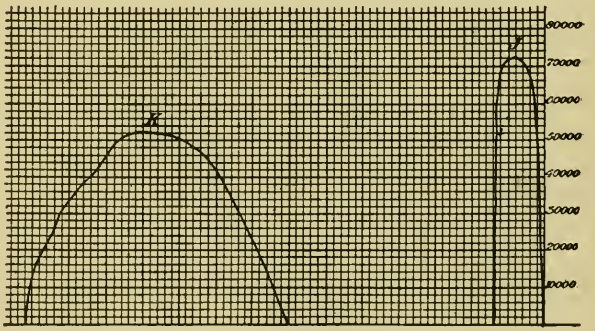


Fig. 3.

cator of the apparatus as used, preferably when steam is used in the punching cylinder, is evident by inspection of the drawing. The drum upon which the pencil works is marked "19."

Figs. 2 and 3 show copies of forms of curves upon specimens of $\frac{7}{16}$ in. thick steel, with the exception of sample "*C*," which was $\frac{5}{16}$ in. thick; the test curves, marked "*A*," "*B*" and "*C*," are of soft steel; "*D*," "*F*" and "*I*" are harder grades of steel; "*C*" is of the softest

basic steel, about 46 000 lbs. tensile strength per square inch; "*B*" of 50 000 lbs. tensile strength, and "*A*" of 69 000 lbs.; "*D*" is of 78 000 lbs. tensile strength steel; "*F*" of 85 000 lbs. tensile strength, and "I" of 75 000 lbs. tensile strength. In Fig. 3, the pencil was controlled by a spring, forming re-current curves; specimen "*J*" being of 78 000 lbs. tensile strength with 14% elongation in 8 ins., sample "*K*" being 65 000 lbs. tensile strength with 24% elongation.

By this method, tests can readily be made upon crop ends of pieces rolled from each ingot of steel used in any given lot of material; or, if necessary, pieces can be tested from crop ends taken from each end of large plates or bars, or from the bars and plates themselves. This ease of procuring samples and making the tests, and the absence of any large special expense therein, constitute a marked practical advantage for this method of testing.

In fact, it will be perfectly practicable to equip the punches used at bridge and boiler shops in actual practice, with some device for measuring the force or work done in punching; and by the aid of graphical devices, a record can be kept of the characteristics of every piece of metal punched that goes into any structure. In this way, it is the writer's judgment, that a more accurate and satisfactory system of testing can be carried on than is possible with methods in general use at present.

It is not to be inferred by this that the need of careful inspection at the mills will be done away with by the adoption of this method of testing; on the contrary, with the general adoption of steel to replace wrought iron in structures, there is, and probably will remain, a constant necessity for careful supervision by unprejudiced and skillful inspectors employed by and in the interests of the purchaser and not of the mills themselves, in order to insure suitable material, and this, too, at the largest and best appointed and managed steel mills of the country, as well as the less experienced and smaller plants. This opinion is expressed after an extended experience with the methods, shop practice and character of the material turned out at most all of the structural steel works of the country.

The characteristics of the metal actually developed by this method of testing are:

1. The ductility of the metal as compared with its tensile and shearing strength.

(*a*) The results of experiments of Appendix A show that with soft, ductile structural steel, with from 50 000 to 70 000 lbs. per square inch tensile strength, the shearing strength of the metal is about 75% of the tensile strength for single shear and about 73% for double shear.

(*b*) In punching tests, the product of the circumference of the punch by the thickness of the section is the area subject to the shearing action of the punch, at least up to $\frac{5}{8}$ in. thick sections. Using this area as a divisor and the actual force applied in pounds to punch the specimen as a dividend, will give as a quotient, with approximate accuracy, the shearing strength per square inch of the specimen.

The experiments described in Appendix B show that with soft, ductile structural steel of from 50 000 to 70 000 lbs. per square inch tensile strength, by using the formula $0.75\ U = S = \frac{P}{c \times t}$ where $U =$ ultimate tensile strength per square inch, $S =$ shearing strength per square inch, $P =$ pressure applied in pounds to punch the specimen, $c =$ circumference of the punch, and $t =$ thickness of the specimens, both the tensile and shearing strength are obtained within a reasonable limit by this method of testing.

(*c*) The experiments of Appendix C show that with very strong but ductile steel, of high tensile strength and large elongation, the formula given in paragraph (*b*) does not apply, as the shearing strength is not over 70%, and in some cases not over 60%, of the tensile strength.

(*d*) The experiments of Appendix D show that with hard and brittle steel with low ductility, the shearing strength more nearly approaches the tensile strength, and in the very hard steel exceeds the tensile strength, so that in these cases the formula given in paragraph (*b*) does not apply.

(*e*) The experiments of Appendix E show that with weak and rotten steel of low tensile strength and ductility the relation of shearing to tensile strength is not uniform, and that when the force applied to make the punching test is either abnormally low or high, or irregular, there is cause for further investigation of the metal.

(*f*) The experiments of Appendix F show that when in punching tests the thickness of the specimen is over $\frac{5}{8}$ in., with soft ductile steel, the effect of crushing instead of shearing is seen on the last or lower portion of the metal of both the punchings and of the walls of the steel in which the holes are punched, and that this factor is liable to

give irregular and misleading results as to quality. In these cases, a double set of punches, working against or into each other, will perhaps correct the trouble.

(*g*) The experiments of Appendix G show that the action of speed in punching does not modify the results to any considerable extent, especially where reasonable care is taken to have the testing conducted under uniform conditions in this respect.

(*h*) The experiments of Appendix H show that variations in power required in punching, due to the location of the punched holes and their environment, while being considerable, as shown by the tables, are nothing like as great as those occasioned by difference in quality of the metal. No marked error need occur in this method of testing, due to the very slight differences in the locations of the centers of the punched holes as regards the edge of the plate or the location of adjoining holes.

2. Experiments show that a method of selecting good structural steel, and discarding that which is unsuitable, can be devised by means of this method of testing its quality.

From the record of the experiments given (and in accordance with the writer's experience), it will be seen that good steel within the limit of 55 000 to 65 000 lbs. tensile strength per square inch, with an elongation of over 23% in 8 ins., a percentage of reduction of area of over 45%, and which pulls apart in tensile tests with a silky fracture, will require for ¾-in. punched holes (with centers 1½ ins. from the edge of the specimen, when the specimen is punched in between 30 and 60 seconds time by a force applied gradually with a punch having $\frac{1}{8}$ in. clearance), a force proportioned to the thickness of the plate or bar, as follows:

Thickness.								
$\frac{3}{16}$	in.	section,	between	20 000	and	24 000	lbs.	pressure.
$\frac{1}{4}$	"	"	"	25 000	"	30 000	"	
$\frac{5}{16}$	"	"	"	33 000	"	40 000	"	
$\frac{3}{8}$	"	"	"	40 000	"	46 000	"	
$\frac{7}{16}$	"	"	"	48 000	"	55 000	"	
$\frac{1}{2}$	"	"	"	56 000	"	63 000	"	
$\frac{9}{16}$	"	"	"	65 000	"	73 000	"	
$\frac{5}{8}$	"	"	"	73 000	"	80 000	"	
$\frac{11}{16}$	"	"	"	80 000	"	88 000	"	
$\frac{3}{4}$	"	"	"	85 000	"	95 000	"	

Similarly, the work done in punching a ¾-in. hole, under similar conditions to those stated above, can be measured with various thickness of section of structural steel, and the specifications may require that it shall be at least equal to the minimum amount in the following table, where the force used as one factor in obtaining the work shall be, as a minimum, in accordance with the results given in the corresponding column:

	Minimum work in inch pounds must be	Where the force applied shall be at least
¼-in. section	3 125 lbs.	25 000 lbs.
5/16 " "	8 250 "	33 000 "
⅜ " "	10 000 "	40 000 "
7/16 " "	12 000 "	48 000 "
½ " "	14 500 "	56 000 "
9/16 " "	17 500 "	65 000 "
⅝ " "	19 000 "	73 000 "
11/16 " "	23 000 "	80 000 "
¾ " "	25 000 "	85 000 "

In another way, the work done in punching ¾-in. holes, under conditions similar to those given before, can be used as a means of specifying suitable structural steel, as shown by the graphical methods of representing the work done in punching in Fig. 3; by stipulating that under given conditions the work done, as measured by the area of cards produced under given specified conditions for each given thickness, shall have a minimum area in square inches equal to given amounts when the vertical height of the figures produced in measuring the work are no more than given amounts.

The writer has no doubt but that other and perhaps better methods will be devised for measuring the force or work applied in punching, cutting, indenting, or drifting the samples to be tested, and undoubtedly the limits advised in this paper for the requirements for specification will have to be revised as more experience is gained and more workers give their attention to the problems involved. The writer by no means wishes to claim that either perfection or the most accurate results have been arrived at in the matter of devising the best methods of applying this suggested new method of testing; on the contrary, better quick-acting machines, where the friction has been balanced or reduced to a minimum by special mechanism, and where new means

entirely may be employed to measure and record the results as well as to perform the several operations will undoubtedly be invented and adopted, should the method become of general use; and, indeed, while the results given in the appendices following this paper are by no means all the results in each line which have been made, they have been chosen as typical results of the entire series. At the same time the writer freely acknowledges that it will take a large number of tests and more workers in the field to give a sufficient amount of data upon which to generalize or to give results which can be safely recommended for specifications upon which important work is based, in the selection of steel for engineering purposes.

The writer's only claim is, that the general principles involved in this proposed method of testing are of more quick application than any other, and indicate and measure in perhaps a better way the qualities in structural steel that are desirable, as well as the undesirable attributes that the material may have which would make it unsuitable to be used for its designed purpose.

The writer does not claim that by the use of the proposed formula the method will give in all cases the tensile strength of the metal, but rather that it suggests a method of testing for that combination of strength and ductility desirable in structural steel. It does not, for instance, necessarily distinguish steel of over 65 000 lbs. from that of a higher tensile strength unless the ductility is correspondingly low. Steel of 70 000 lbs. tensile strength, with 27% elongation in 8 ins., might be accepted by the method of testing suggested, in the place of a maximum 65 000-lb. steel of some specifications, on account of the exceedingly good ductility of the specimen referred to; though such a steel could not be accepted under this method of testing when the work is used as the crucial test, and the stipulation is made that the work shall be at least a minimum amount, the force as one factor of the work not being over a given stipulated amount. But steel with 70 000 lbs. tensile strength with only 18% elongation in 8 ins. would be rejected, as shown by comparing the demands of the foregoing tables for proposed specifications for structural material, with the results from such steel as shown in the table of Appendix D.

This, the writer believes, is as it should be. A higher tensile strength steel, if accompanied with good ductility, as measured by stretch and reduction in tensile tests, should be accepted and not re-

jected, and the fact that this method of testing would include such steel is, in the writer's judgment, a point in favor of this method of testing.

Again, where the section is over $\frac{5}{8}$ in. in thickness, the crushing strength seems to be exceeded in the bottom portion of the punched metal, and the walls of both the hole and punching show markedly the line of demarcation between the sheared and the crushed surfaces of the metal. The force applied in punching seems to vary much more with steel of over $\frac{5}{8}$ in. thickness for varying factors of speed of the punching operation, character of the punch, and similar occasions for the variance of the results of the punching test; and the author does not claim from his experience that the same degree of accuracy can be obtained by this method of testing sections of over $\frac{5}{8}$ in. thickness as with thinner sections with the forms of apparatus already devised.

With the results given in Appendix G, it is shown that the speed with which the punching operation is done has only a comparatively slight influence upon the results. The force required to punch a given-sized hole through a given thickness of the same steel being a few thousand pounds more when the force is applied upon the punch quickly than when the force is applied gradually.

This variation, however, is not more than 2 000, or at most 3 000, lbs., with material up to $\frac{3}{8}$ in. thickness, and is no more than 5 000 for steel of $\frac{5}{8}$ in. thickness. These differences are well within the variance that should be allowed for good material, and, as our experience has shown, are nothing like the differences shown by the methods of testing between good, soft, ductile steel and hard or brittle steel.

The results as given in the tables of Appendices B, C and G show the difference between punching $\frac{3}{4}$ and $\frac{7}{8}$ in. diameter holes through different thicknesses of steel, and the results given in Appendix H show that where the location of the center of the punch varies in distance from the sheared or rolled edge of the steel it gives comparatively small variations in results. This table indicates that with ordinary care, precaution being taken to have the tools uniform in character and the methods of working reasonably similar, no wide variance or discrepancy of results will be encountered.

The same remark also applies to the method of dressing or arming the cutting edge and end of the punching or cutting tools. Widely varying ways of doing this will produce marked differences in the force

applied or work done in cutting or punching, through equivalent sections of the same steel. At the same time the differences due to these varying conditions are not so marked nor the method of testing so sensitive as to render the results untrustworthy when ordinary precautions necessary to accuracy with any method of testing are observed.

An advantage of the system is that it will always be perfectly practicable to keep, at places where tests are made, standard specimens of the thicknesses generally worked upon, which have been tested by other machines and under other conditions, and of which full records of results are ready for comparison; and at any time questions as to the apparatus in use being out of order or giving discordant results can be readily proven by making tests with the standard specimens to check the working of the apparatus.

When the necessary precautions that have been explained in this paper have been taken, it is the writer's judgment that this system of testing the quality of steel can be safely and conveniently used for structural purposes, and that it is sensitive enough and will exclude all classes of bad steel, and that which is unsuitable. It will, of course, take a large amount of testing and experience to develop all the facts regarding this. In a general way, however, the writer believes that, especially where specifications for structural steel are drawn requiring a minimum work where the actual force applied as one of the factors of the work is within given limits for each varying thickness of section, the results obtained by the method can be safely relied upon to reject, not only steel that will be too hard or too soft, but also that which will be too brittle or too weak, or that which has been burned, or steel that has not had a sufficient amount of "work" —in short, that it will reject metal that has any of the injurious qualities that steel is ordinarily rejected for, as regards its physical properties.

APPENDIX A.

Determination of Relation of Shearing to Tensile Strength.

Samples of $\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$ and $\frac{7}{8}$ in. diameter round rods of soft steel were taken by the writer for experiments, care being taken in each instance to know the rods were each from homogeneous and uniform melts of steel that had previously been ascertained by frequent tests to be of good representative quality of that used in ordinary structural work, most of it being of a character used in the fabrication of rivets.

One or more portions of each rod were pulled apart in the 100 000-lb. Olsen testing machine of the Pittsburgh Testing Laboratory, and the ordinary tensile test results obtained are recorded in Table A of this appendix.

Adjoining portions of the same rods were sheared apart, either in single or double shear, in an apparatus in use by the writer at the Pittsburgh Testing Laboratory, and which is shown in Figs. 5, 6, 7 and 8. The opposite ends of the apparatus were, in most instances, held between the upper fixed and lower movable cross-heads of the Olsen testing machine of the Pittsburgh Testing Laboratory, although corroborative check results were obtained by use of the same apparatus attached to the heads of an 100 000-lb. Emery machine.

Single Shear

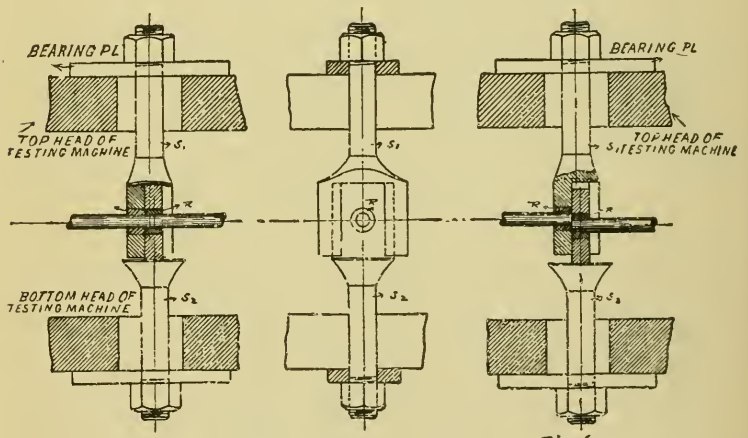


Fig. 5. Fig. 6.

The bearings or cutting edges of the shearing apparatus are rings R of Figs. 5, 6, 7 and 8 and are of specially prepared hard tool steel. Concordant, and the writer believes accurate, results were obtained in all the tests recorded in Table A, great care being taken to insure the specimens being sheared directly in line without unnecessary bending moments, and with precautions to keep friction at a minimum.

The results indicate that somewhat greater shearing strength is shown in single than in double shear with the method of testing adopted. An average of 16 results of specimens Nos. 3 to 18 inclusive, give the shearing strength in single shear as 75.46% of the

ultimate tensile strength, and only 73.56% in double shear. This is probably due to increased bending moments in the case of the specimens subject to double shear. Still, as the rods very closely filled the holes in the cutting rings in each instance, and the distance between shearing planes was 1 in., the conditions were very similar to the strains of double shear produced on rivets in actual practice.

The results, as a whole, show that the usual stipulations in engineering specifications, and the practice of allowing a unit strain in structural steel subject to shearing stress of 80% of that allowed in tension, is probably too liberal, and that it should be reduced to about 70% in metal subject to double shear, and not over 75% in that subject to single shear.

Double Shear

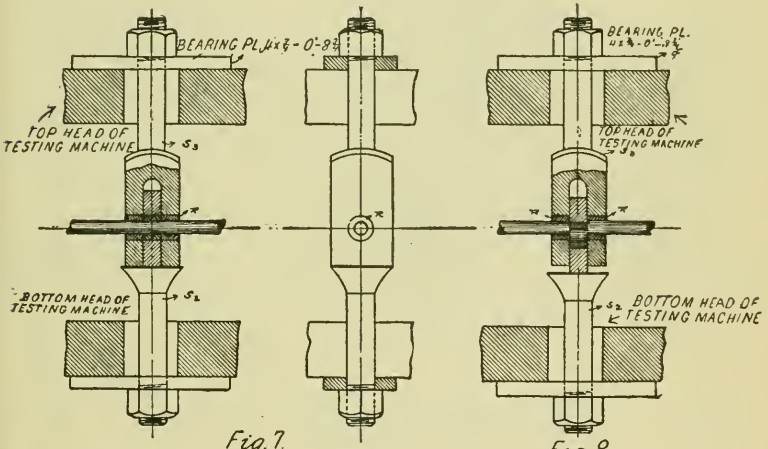


Fig. 7. Fig. 8.

Unfortunately, the literature of the relation of shearing to tensile strength is very limited, and the exact relations under different conditions have not yet been thoroughly worked up, so far as the writer can find in the various works on "Mechanics and Strength of Materials," which he has consulted. Nearly all of the works seem to agree that the shearing strength is 80% of the tensile, but very few results of actual experiments are given to substantiate these figures.

Thomas Box, in his work, deduces his values of the pitch of rivets by considering the shearing strength of the rivet per square inch as equal to 75% of the tensile strength of the plate per square inch.

Lanza, in his admirable work on "Applied Mechanics," paragraph 229, quotes the shearing strength of wrought iron and steel, as deduced from an experiment on torsional strength as: "Found to be about equal to the tensile strength. When deduced from experiments on riveted joints, it is found to be about three-quarters or four-fifths of the tensile strength."*

APPENDIX B.

A punching apparatus was used for the major portion of the tests given in Table B, and, in fact, was used in the punching results of the other tables of Appendices B, C, D, E, F, G and H as shown in Fig. 4.

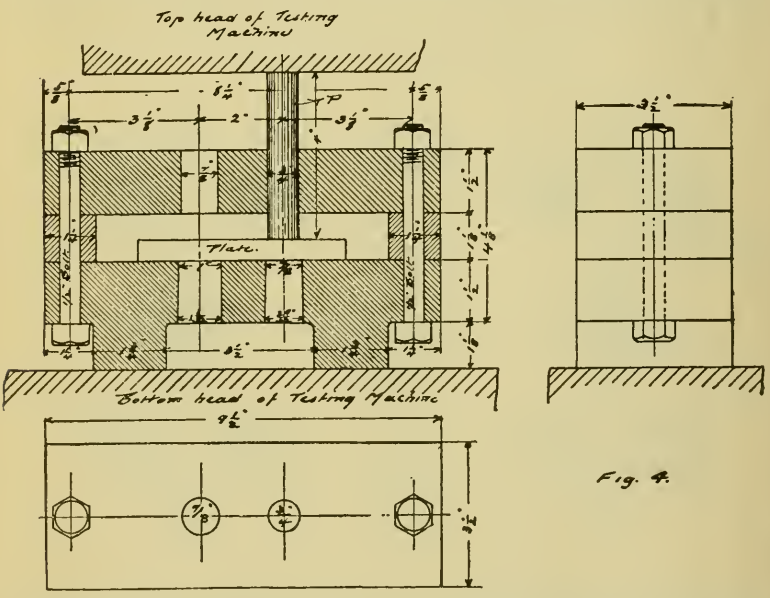


Fig. 4.

It consists of two plates of steel 1½ ins. thick, which are held apart by 1⅛-in. blocks and bolted securely together with two ½-in. bolts at either end. The length of the apparatus is 9½ ins., the base plate having 1¾-in. projections upon which the apparatus rests. The plates are pierced with two holes for a ¾-in. punch in one instance, and a ⅞-in. punch in the other. The diameter of the lower dies is from 1 to $1\frac{1}{32}$ ins. for the ⅞-in. punch, and from ⅞ to $\frac{29}{32}$ in. for the ¾-in. punch.

* Very valuable data upon the shearing strength of steel has been lately published in the *American Engineer and Contract Journal*, by Mr. H. V. Loss; the writer having gone into the subject of shearing strength very fully.

TABLE A.

1	2	3	4	5	6	7	8	9	10	11	12	13
			Tensile Test.				Shearing Test.					
Specimen Number.	Description.	Diameter.	Elastic limit per square inch.	Ultimate strength per square inch.	Elongation in 8 ins. Percent.	Reduction of area. Percent.	Pounds actual exerted in single shear.	Pounds actual exerted in double shear.	Pounds per square inch exerted in single shear.	Pounds per square inch exerted in double shear.	Percent. of single shear to tensile strength.	Percent. of double shear to tensile strength.
1*a*	⅝″ rod.	0.625	.....	57 910	.....	57.62	.....	.....	44 330	.....	76.55	
1*b*	"	0.625	.....	57 910	.....	57.62	.....	.....	45 310	.....	78.24	
2*a*	"	0.625	.....	56 780	.....	62.17	.....	.....	44 000	.....	77.50	
2*b*	"	0.625	.....	56 780	.....	62.17	.....	.....	43 840	.....	77.21	
3	⅞″ rod.	0.858	43 590	61 400	30.50	59 52	27 100	52 250	46 870	45 180	76.34	73.60
4	"	0.857	43 250	58 770	31.25	60.30	26 900	51 450	46 640	44 600	79.33	75.90
5	"	0.860	42 520	60 170	30.00	60.00	26 700	52 000	45 960	44 760	76.38	74.39
6	"	0.855	42 240	60 970	30.00	60.84	26 200	51 600	45 640	44 950	74.86	73.56
7	¾″ rod.	0.745	44 970	66 300	29.25	51.46	20 450	42 350	46 910	48 580	77.75	73.28
8	"	0.745	44 960	66 070	25.00	57.98	20 200	42 250	46 340	48 410	70.15	73.26
9	"	0.743	42 900	61 810	27.75	58.44	20 000	39 200	46 130	45 200	74.68	73.13
10	"	0.756	42 990	61 700	29.50	61.35	21 550	41 700	48 010	46 450	77.81	75.94
11	⅝″ rod.	0.642	42 010	60 550	27.50	63.11	14 700	31 100	45 410	48 040	75.00	79.35
12	"	0.642	42 010	60 860	29.50	62.52	14 100	30 150	43 560	46 570	71.58	76.52
13	"	0.634	43 390	56 700	22.50	59.20	14 700	27 450	46 560	43 470	82.12	77.55
14	"	0.634	40 220	56 060	28.75	58.18	14 650	27 650	46 400	43 790	82.76	78.11
15	½″ rod.	0.510	48 950	65 100	22.00	44.78	9 700	17 550	47 470	42 950	72.92	65.98
16	"	0.509	46 680	64 870	27.00	42.21	9 400	17 750	46 190	43 620	71.21	67.24
17	¾″ rod.	0.756	44 550	61 040	27.00	62.98	20 950	40 950	46 670	45 610	76.48	74.80
18	"	0.757	43 550	60 880	27.50	58.07	20 550	40 700	45 660	45 220	74.84	74.26
19	⅞″ rivet.	0.875	.....	.....	.....	.....	27 200	.....	45 230			
20	"	0.878	.....	.....	.....	.....	24 500	.....	40 460			
21	"	0.865 S.S. 0.871 D.S.	.....	.....	.....	.....	26 100	53 200	44 410	44 640		
22	¾″ rivet.	0.752	.....	.....	.....	.....	19 300	.....	43 470			
23	"	0.764	.....	.....	.....	.....	19 000	.....	41 450			
24	"	0.761	.....	.....	.....	.....	22 000	.....	48 380			
25	"	0 767	.....	.....	.....	.....	20 800	.....	45 020			
26	⅝″ rivet.	0.637	.....	.....	.....	.....	14 650	.....	45 960			
27	"	0.636	.....	.....	.....	.....	14 150	.....	44 540			

As the apparatus is arranged in Fig. 4, the plate to be punched (marked "Plate") is placed in position and the $\frac{3}{4}$-in. punch is placed upon it, the whole apparatus setting upon the bottom plate of the testing machine, from which the pressure applied is transmitted through to the weighing apparatus of the machine, the top head of the machine pressing directly upon the top of the $\frac{3}{4}$-in. punch. The other general dimensions are shown by inspection of Fig. 4.

The power required in punching samples of good structural steel is shown in Table B of this appendix. The specimens were selected as those having a tensile strength of between 50 000 and 70 000 lbs. per square inch, with good ductility.

Punching tests were made from the same material, with both the $\frac{3}{4}$ and $\frac{7}{8}$-in. punches in most instances. The power was applied and measured, in these tests of Table B, upon an Emery 100 000-lb. capacity testing machine in all cases, except samples marked "6," "8," and "9," in which the first and second results, marked "a" and "b," were measured on the Emery testing machine, and the third and fourth marked "c" and "d," on a 100 000-lb Olsen testing machine of the Pittsburgh Testing Laboratory. The legend of this table and, indeed, of the following tables of the other appendices of this paper, is as follows :

U = ultimate tensile strength per square inch.

S = shearing strength per square inch, as obtained by getting 75% of the tensile strength.

P = pressure applied in pounds to punch the material.

c = average circumference of the punching.

t = thickness of the specimen punched.

The shearing strength per square inch is represented by the formula:

$$\frac{P}{c \times t}.$$

Due to the large clearance of the punching apparatus, the value of c, or the average circumference, was found to be very closely the same through the varying thicknesses of metal, and was taken for the $\frac{3}{4}$-in. punch to be 2.5133 ins. ; and with the $\frac{7}{8}$-in. punch, c, or the average circumference, was taken at 2.9217 ins.

By inspection of Table B, the increment due to increased thickness to produce the punchings is shown throughout to be more or less regular, although the increment varies more above the thickness

of $\frac{5}{8}$ in. than with thinner sections, as shown more clearly in Appendix F. The average increment of pressure is found to be, as an average of a large number of experiments, from 112 to 120 lbs. for each $\frac{1}{1000}$ in. added thickness for the $\frac{3}{4}$-in. diameter punchings, and from 133 to 137 lbs. similarly for the $\frac{7}{8}$ diameter punchings in ordinary cases of good steel.

Specimen No. 1 is acid open hearth steel.
" 2 " " " "
" 3 " " Bessemer "
" 4 " basic open hearth "
" 5 " acid Bessemer "
" 6 " basic open hearth "
" 7 " " " "
" 8 " " " "
" 9 " " " "
" 10 " " " "
" 11 " acid " "
" 12 " " " "
" 13 " " Bessemer "
" 14 " basic open hearth "
" 15 " acid " "
" 16 " " " "
" 17 " " Bessemer "

Specimen No. 18 is acid Bessemer steel.
" 19 " " " "
" 20 " " " "
" 21 " " open hearth "
" 22 " " Bessemer "
" 23 " " open hearth "
" 24 " basic " "
" 25 " acid " "
" 26 " " " "
" 27 " basic " "
" 28 " " " "
" 29 " " " "
" 30 " acid " "
" 31 " " " "
" 32 " " " "
" 33 " basic " "

APPENDIX C.

Similar tests to those described in Appendix B were made by the writer upon similar samples of steel of extra purity and quality, and which have exhibited very high ductility as compared to the ultimate strength in tensile tests.

The same legend which has been used in Appendix B applies to this Appendix and its Table C.

The results of 12 tests upon six different samples give, as an average of the percentages of the shearing strength per square inch of the ultimate tensile strength, 67.63% for the steel punched with $\frac{3}{4}$-in. punch, and 67.74 % with the same steel punched with the $\frac{7}{8}$-in. punch.

All the specimens of steel of Table C were made by the acid open hearth process.

The results clearly show that with this class of steel the shearing strength per square inch, as determined by the formula $\frac{P}{c \times t}$, is a smaller percentage of the ultimate tensile strength, and the steel is not of so great comparative ductility, a state of affairs that by no means points necessarily to a poor quality of steel, and, in fact, in the par-

ticular steel under consideration, is an evidence of superior quality instead.

APPENDIX D.

Similar tests to those described in Appendices B and C were made by the writer upon samples of hard and brittle steels, most of which owed their hard and brittle qualities to high carbon or high phosphorus, or a combination of the two, or to large proportions of manganese probably not uniformly distributed through the metal.

The same legend which has been used in the previous Appendices B and C applies to this appendix and its Table D.

In the case of the hard and brittle steels, the results given in Table D show that the shearing strength approaches much more nearly the tensile strength than with softer and more ductile metal.

Specimens numbered 1 to 5 inclusive were of acid Bessemer steel. Specimen No. 6 was made by the basic Bessemer process ; and specimen No. 8 was crucible steel.

APPENDIX E.

Similar tests to those described in Appendices B, C, and D were made by the writer upon samples of steel which had proven weak and rotten in tensile tests.

The same legend which has been used in the previous Appendices B, C and D applies to this appendix and to Tables E1 and E2.

The proportion between the shearing strength as determined by the formula $\frac{P}{c \times t}$ and the ultimate tensile strength, varies greatly with this class of steel, due to the peculiar condition it may be left in. Some weak steel is superficially quite hard, while others are softer throughout than ordinary steel.

In almost all cases the pressure applied is less in punching with this class of steel than with good steel, though this is not, in fact, the case with the samples of weak steel chosen for this appendix.

In the case of the "burned" steel of ordinary grades, Table E2 shows an average loss of $2\frac{36}{100}\%$ for $\frac{3}{4}$-in. punching, and $1\frac{48}{100}\%$ with the $\frac{7}{8}$-in. punch, of the power used in punching the same metal before it was burned.

All the specimens of Table E2 were the same as previously reported

in Table B, the steel being purposely and markedly "burned" for the purpose of making these tests.

The writer's experience with testing this class of metal points to the necessity of the use of some method of measuring the work done in punching, as a crucial test by this method for pointing out weak and rotten steel, where the work done instead of the force alone is employed as the method of testing. It seems very easy to discriminate this class of poor steel, as the amount of work done is always markedly less in such cases than with good steel treated similarly.

In the case of specimens of Table E1, the steel had evidently never been properly refined, due in the case of specimens Nos. 4, 5, 6 and 7, made by the acid open hearth process, to a cold open hearth furnace, unskillful melting, failure of gas supply, or a similar cause. In the case of the acid Bessemer steel specimens Nos. 1 and 3, due to a bad pig-iron mixture working cold, a lack of wind pressure or similar cause.

In the cases of burned steel shown in Table E2, often such steel is rendered superficially hard and brittle, probably due to the absorption by the metal of considerable amounts of occluded gases at the high temperature it had been subjected to at the time it was burned. Most steels, however, show a less tensile strength and take less power to punch after being burned. In other words, these burned steels are weaker and are made rotten by burning, as shown by inspection of Table E2. In some cases, however, probably, as above explained, due to the occlusion and chemical combination of occluded gases, burned steel is rendered hard and brittle, although in such cases the writer's experience has been that the elastic limit of tensile specimens very closely approaches the ultimate strength, and by the burning operation the character of the steel is made to approach that of cast iron.

By inspection of the pressures applied upon the specimens of the steel, both before and after burning, a marked diminution of pressure will be noted in the burned specimens, and had the results been calculated upon the same thicknesses of metal, columns 12 to 17, inclusive, of Table E2 would have shown the same marked variances, so that in actual practice burned steel of any given section can be told ordinarily by the diminution in pressure required in punching. The burning operation in each case raised a heavy scale which was cleaned off before the specimens were punched in each case, and the calculations of the table were made upon the decreased sections of actual metal punched.

APPENDIX F.

Action of the Thickness of the Specimen in Punching.

The records of tests of steel of over $\frac{5}{8}$ in. thickness so far made by the writer have not given as uniform results as similar results of steel lower thickness.

There seems to be a crushing action upon the lower sections of the metal which seems to be plainly shown upon examination of both the walls of the punched holes and the punchings themselves.

The tests have also shown the general shop rule to be a good one, of never using a punch in steel of a thickness greater than the diameter of the punch. In several instances of punching up to $\frac{7}{8}$ and 1 in. thickness, the punches of $\frac{3}{4}$ in. diameter made of best steel failed, in most cases, by bending.

As a general rule, material thicker than $\frac{5}{8}$-in. requires a little lower pressure per square inch than thinner sections of the same steel, probably due partly to the crushing action of the lower part of the metal mentioned above, and partly to the fact that the thicker steel has not had proportionately so much work upon it in most cases, and is not so strong, either in tensile or shearing strength, as the thinner rolled material.

Specimens, the tests of which are tabulated in Table F as Nos. 1, 2, 10, 11, 13, are of acid Bessemer steel; Nos. 3, 5, 8 are of basic open hearth steel; and Nos. 4, 6, 7, 9, 12, 14 are acid open hearth steel.

APPENDIX G.

The Action of Speed of Punching in Modifying the Results in Punching Tests.

A considerable number of specimens of steel were taken which had been proven to be very homogeneous throughout, and on which a good many results of punching tests had been obtained. These were again tested and the power applied in punching both $\frac{3}{4}$ and $\frac{7}{8}$-in. holes was measured. The power was applied very gradually, from 1 to 10 minutes' time being occupied in some cases, and in others, on the same steel, performing the punching in from 10 to 60 seconds. This was done very accurately, with the aid of the rapid-working plunger of an

TABLE B.

Samples of Good Soft Steel of Average Quality between 50 000 and 70 000 Lbs. Tensile Strength per Square Inch.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
			Tensile Tests.				Punching Tests.		Results Obtained by Calculation.					Analyses of Steel.		
Specimen Number.	Description cut from—	Thickness of specimen.	Elastic limit per square inch.	Ultimate strength per square inch. U.	Elongation in 8 ins. Percent.	Reduction of area. Percent.	Pressure applied in pounds to punch $\frac{3}{4}$-in. diameter punch. P.	Pressure applied in pounds to punch $\frac{7}{8}$-in. diameter punch. P.	$\frac{P}{c\,x\,t}$ with $\frac{3}{4}$-in. diameter punch. $c = 2.5133$.	$\frac{P}{c\,x\,t}$ with $\frac{7}{8}$-in. diameter punch. $c = 2.9217$.	$S = 75\%$ of U	Percent. $\frac{P}{c\,x\,t}$ is to ultimate strength, $\frac{3}{4}$-in. diameter punch.	Percent. $\frac{P}{c\,x\,t}$ is to ultimate strength, $\frac{7}{8}$-in. diameter punch.	Carbon. Percent.	Manganese. Percent.	Phosphorus. Percent.
1 a	$6\frac{3}{4}$" x $\frac{3}{16}$" Pl.	0.185	48 360	65 720	20.00	57.04	21 600	25 200	46 450	46 610	49 290	70.68	70.92	0.12	0.50	0.064
1 b, c	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
2 a	2" x 2" x $\frac{7}{16}$" L.	0.187	46 200	66 200	21.30	58.10	23 500	27 500	60 000	50 320	49 650	75.54	76.10	0.22	0.51	0.021
2 b	"	"	"	"	"	"	"	"	"	"	"	75.52	"	"	"	"
3 a	3" x 3" x $\frac{1}{4}$" L.	0.245	41 770	61 590	25.00	51.90	28 300	31 800	45 960	44 430	46 190	74 64	82.74	0 11	0.46	0.086
3 b	"	"	"	"	"	"	"	32 200	"	44 980	"	"	73.04	"	"	"
4 a	15" x $\frac{1}{4}$" Pl.	0.256	42 200	63 500	25 80	56.30	29 800	33 500	46 320	44 780	46 630	72,94	70,52	0.24	0.44	0.018
4 b	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
5 a	3" x 3" x $\frac{5}{16}$" L.	0.310	38 900	60 970	25.00	53.80	37 000	41 300	47 500	45 830	45 730	77.91	75.16	0.10	0.46	0.084
5 b	"	"	"	"	"	"	36 800	"	47 240	"	"	77.48	"	"	"	"
6 a	62" x $\frac{5}{16}$" Pl. 136"	0.320	37 200	61 590	24.80	64.20	36 500	40 800	46 280	43 640	46 190	73.68	70.86	0.25	0 48	0.023
6 b	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
6 c	"	"	38 000	62 400	25.80	65.00	37 100	43 600	46 130	44 170	46 800	73,94	70.78	"	"	"
6 d	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
7 a	$10\frac{1}{2}$" x $\frac{3}{8}$" Pl	0.365	38 630	58 370	25.00	62,50	45 700	46 800	49 810	43 880	43 780	85,32	75.16	0.15	0.45	0.012
7 b	"	"	"	"	"	"	44 600	48 900	47 960	45 850	"	82,12	78.54	"	"	"
8 a	10" x $\frac{3}{8}$" Pl.	0.365	41 960	67 170	26.25	51.47	46 100	53 800	50 250	50 440	50 380	74 80	75.10	0.24	0.45	0.028
8 b	"	"	"	"	"	"	45 800	53 000	49 920	49 700	"	74.32	73.99	"	"	"
8 c	"	"	42 500	68 000	26.80	50,90	"	53 900	50 040	50 640	51 000	74.50	75.24	"	"	"
8 d	"	"	"	"	"	"	"	"	49 920	"	"	74.32	"	"	"	"
9 a	10" x $\frac{3}{8}$" Pl.	0.364	39 060	64 240	26.25	50,62	46 600	52 400	50 930	49 270	48 180	79.28	76.70	0.27	0.42	0.017
9 b	"	"	"	"	"	"	46 300	52 100	50 600	49 010	"	76.29	76.24	"	"	"
9 c	"	"	38 200	63 500	26.00	51.30	46 600	52 000	50 270	48 900	47 630	76.12	77.00	"	"	"
9 d	"	"	"	"	"	"	46 100	51 800	50 380	48 710	"	75.82	76.70	"	"	"
10 a	10" x $\frac{3}{8}$" Pl.	0.370	32 100	50 800	29.80	72,10	39 800	45 100	42 800	41 720	38 100	84,26	82,10	0.13	0.29	0 018
10 b	"	"	"	"	"	"	40 200	45 300	43 230	41 900	"	85,90	82 48	"	"	"
11 a	"	0.360	42 820	70 790	23.75	51.70	48 100	54 500	53 160	51 310	53 090	75,10	73.21	0.19	0.49	0 071
11 b	"	"	"	"	"	"	"	54 800	"	52 100	"	"	73.60	"	"	"
12 a	25" x $\frac{7}{16}$" Pl.	0.439	40 350	61 910	28.00	53,80	50 200	58 300	47 370	46 410	46 440	76,50	74,98	0.15	0,33	0.074
12 b	"	"	"	"	"	"	51 600	58 400	47 330	46 490	"	77.10	75.09	"	"	"
13 a	4" x 4" x $\frac{7}{16}$" L.	0.438	42 000	63 500	24,60	50,30	54 000	62 100	49 650	48 530	47 630	77.24	76.43	0.14	0 43	0.081
13 b	"	"	"	"	"	"	53 800	61 800	48 870	48 290	"	76.36	76.05	"	"	"
14 a	20" x $\frac{1}{2}$" Pl.	0 503	39 360	61 990	26 00	52,30	60 300	66 900	47 980	45 790	46 490	77.27	73.88	0 21	0.42	0.011
14 b	"	"	"	"	"	"	59 400	68 100	47 270	45 240	"	77.26	73.99	"	"	"
15 a	16" x $\frac{1}{2}$" Pl	0.505	37 300	55 900	27,30	64,20	56 900	64 800	44 840	43 920	41 920	80.21	78.56	0.10	0.36	0.073
15 b	"	"	"	"	"	"	57 200	65 300	45 070	44 260	"	80.62	79 17	"	"	"
16 a	5" x $3\frac{1}{2}$" x $\frac{1}{2}$" L.	0.536	39 210	59 900	26.20	58,30	62 000	70 100	46 020	44 770	44 930	76,83	74,74	0.12	0,37	0.062
16 b	"	"	"	"	"	"	62 100	69 400	46 100	44 320	"	76,96	73 99	"	"	"
17 a	6" x 6" x $\frac{7}{16}$" L.	0.580	39 480	62 380	26.25	55 30	70 000	79 000	48 020	46 620	46 790	76.98	74,74	0.15	0.54	0.097
17 b	"	"	"	"	"	"	69 000	78 300	47 340	46 210	"	75,89	74.08	"	"	"
18 a	6" x 6" x $\frac{7}{16}$" L.	0.555	38 710	60 910	25,00	54.40	67 100	76 400	48 100	47 110	45 680	78,97	77.34	0.13	0.32	0.103
18 b	"	"	"	"	"	"	65 700	76 100	47 090	46 930	"	77.32	77.06	"	"	"
19 a	18" x $\frac{7}{16}$" Pl.	0.560	36 430	63 290	28,50	59,93	61 000	71 400	43 340	43 640	47 470	68.48	68,95	0.16	"	0.089
19 b	"	"	"	"	"	"	62 000	71 500	44 050	43 710	"	69,60	69.06	"	"	"
20 a	5" x 5" x $\frac{5}{8}$" L.	0.600	39 440	62 380	25.00	50.70	74 600	83 300	49 070	47 520	46 790	78,66	76.18	0.10	0.50	"
20 b	"	"	"	"	"	"	74 300	83 200	49 270	47 460	"	78,98	76,10	"	"	"
21 a	10" x $\frac{5}{8}$" Pl.	0.620	43 000	69 500	23.80	48.60	73 900	84 600	48 720	46 700	52 130	70.10	67,20	0.18	0,39	0.077
21 b	"	"	"	"	"	"	75 400	86 200	48 400	47 580	"	69,64	68,46	"	"	"
22 a	6" x 6" x $\frac{11}{16}$" L.	0.670	40 700	61 100	23.75	56.20	86 000	93 100	51 070	47 590	45 850	83 58	77.81	0.16	0 69	0 105
22 b	"	"	"	"	"	"	85 500	93 700	50 830	47 870	"	83 25	78.34	"	"	"
23 a	18" x $\frac{11}{16}$" Pl.	0.630	37 500	56 200	24 80	53,90	80 500	89 100	46 420	44 200	42 150	82 20	78.65	0.17	0,49	0.082
23 b	"	"	"	"	"	"	80 200	89 000	46 260	44 150	"	82,32	78.56	"	"	"
24 a	4" x 4" x $\frac{7}{8}$" L.	0.725	33 330	60 160	28,75	54,90	82 000	97 300	45 010	45 930	45 120	74.84	76 34	0.15	0.45	0 009
24 b	"	"	"	"	"	"	83 800	96 700	46 000	45 650	"	76 46	75,88	"	"	"
25 a	10" x $\frac{3}{4}$" Pl.	0 755	37 090	63 300	30.00	62.00	83 000	92 000	43 740	41 670	45 300	72 44	69,00	0.13	0.58	0 030
25 b	"	"	"	"	"	"	85 000	99 000	44 800	44 840	"	74,24	74.25	"	"	"
26 a	10" x $\frac{13}{16}$" Pl	0.817	.....	.....	.....	.....	90 300	97 500	43 980	40 850	.....	.....	.....	"	0,53	0,077
26 b	"	"	.....	.....	.....	.....	88 100	100 400	42 920	42 060	.....	.....	.....	"	"	"
27 a	6" x 6" x $\frac{13}{16}$" L.	0.813	38 200	62 400	24.30	62.00	93 000	101 900	45 520	42 910	46 800	72,96	68,76	0 19	0 37	0.028
27 b	"	"	"	"	"	"	94 000	100 700	46 020	42 400	"	73.74	67 95	"	"	"
28 a	24" x $\frac{7}{8}$" Pl.	0.880	32 650	57 870	27.00	59,00	101 000	114 000	46 640	44 340	43 410	78,86	76.62	0,25	0,51	0.009
28 b	"	"	"	"	"	"	.....	107 000	.....	41 630	"	.....	71.94	"	"	"
29 a	18" x $\frac{7}{8}$" Pl.	0 876	34 000	64 800	23.80	51.10	105 000	119 000	47 680	46 600	48 600	73,59	71 76	0.27	0,37	0.021
29 b	"	"	"	"	"	"	103 000	121 000	46 800	47 290	"	72,22	72,90	"	"	"
30 a	6" x 6" x $\frac{15}{16}$" L.	0,910	.....	.....	.....	.....	.....	119 000	.....	44 760	.....	.....	.....	0.11	0.58	0.092
31 a	"	0 938	34 500	62 100	23,00	48.00	117 000	120 000	49 630	43 800	46 580	79.92	70.54			
31 b	"	"	"	"	"	"	110 000	"	46660	"	"	75,14	"			
32 a	21" x 1" Pl.	1.010	38 270	56 320	30,50	63.30	.....	121 000	.....	41 010	42 240	.....	72,82			
32 b	"	"	.....	.....	.....	.....	.....	"	.....	"	"	.....	"			
33 a	15" x 1" Pl.	1,008	35 630	57 200	23,80	47.90	113 000	124 000	44 620	42 110	42 900	78,01	73.62	0.23	0,39	0.025
33 b	"	"	"	"	"	"	115 000	126 000	45 410	42 780	"	79,38	74.79	"	"	"

TABLE C.

Very Pure and Good Steel of High Tensile Strength and Ductility.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
			Tensile Test.				Punching Test.		Results Obtained by Calculation.				Analyses.		
Specimen Number.	Description cut from—	Thickness of specimen.	Elastic limit per square inch.	Ultimate strength per square inch.	Elongation in 8 ins. Percent.	Reduction of area. Percent.	Pressure applied in pounds to punch with ¾-in. punch.	Pressure applied in pounds to punch with ⅞-in. punch.	$\frac{P}{c \times t}$ with ¾-in. diameter punch.	$\frac{P}{c \times t}$ with ⅞-in. diameter punch.	Percent. of $\frac{P}{c \times t}$ with ¾-in. diameter punch to ultimate strength.	Percent. of $\frac{P}{c \times t}$ with ⅞-in. diameter punch to ultimate strength.	Carbon, Percent.	Manganese, Percent.	Phosphorus, Percent.
1a	10'' × 3/16'' Pl.	0.365	44 220	73 600	24.25	50.67	46 600	52 100	50 800	48 900	69.02	66.37	0.17	0 55	0.041
1b	"	"	"	"	"	"	46 600	53 000	50 800	49 750	69.02	67 53	"	"	"
2a	10'' × 3/16'' Pl.	0.360	40 820	70 790	25.75	51.77	48 100	54 900	53 160	52 200	75.10	73.74	0.18	0.51	0.037
2b	"	"	"	"	"	"	47 900	54 700	52 940	52 000	74.78	73.45	"	"	"
3a	10'' × 3/16'' Pl.	0.367	42 060	70 170	25.75	50.99	47 500	52 800	51 500	49 240	73.39	70.17	0.16	0.49	0.042
3b	"	"	"	"	"	"	46 800	52 900	50 740	49 340	72.31	70.32	"	"	"
4a	5'' × 3½'' × ½'' L.	0.536	41 300	70 200	27.30	54.60	62 100	71 800	46 100	45 850	65.67	65 34	0.19	0.51	0.029
4b	"	"	"	"	"	"	61 800	71 900	45 870	45 920	65.34	65.41	"	"	"
5a	5'' × ½'' bar.	0.505	47 900	82 400	25.10	52.25	64 900	73 400	51 720	49 750	62.04	60.38	0.24	0.48	0.031
5b	"	"	"	"	"	"	65 200	73 500	51 360	49 810	62.33	60.45	"	"	"
6a	6'' × 6'' × 9/16'' L.	0.561	45 000	78 800	26 25	54.00	68 800	76 300	48 790	46 550	61.92	59.07	0.20	0.59	0.035
6b	"	"	"	"	"	"	69 000	76 800	48 940	46 850	*62 11	†59.45	"	"	"

* Average, 67.63 per cent.
† Average, 67.74 per cent.

TABLE D.

Hard Brittle Steel.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
			Tensile Test.				Punching Test.		Results Obtained by Calculation.				Analyses.		
Specimen Number.	Description cut from—	Thickness of specimen.	Elastic limit per square inch.	Ultimate strength per square inch.	Elongation in 8 ins. Percent.	Reduction of area. Percent.	Pressure applied in pounds to punch with ¾-in. punch.	Pressure applied in pounds to punch with ⅞-in. punch.	$\frac{P}{c \times t}$ with ¾-in. diameter punch.	$\frac{P}{c \times t}$ with ⅞-in. diameter punch.	Percent. of $\frac{P}{c \times t}$ with ¾-in. diameter punch to ultimate strength.	Percent. of $\frac{P}{c \times t}$ with ⅞-in. diameter punch to ultimate strength.	Carbon. Percent.	Manganese. Percent.	Phosphorus. Percent.
1a	10″ × 3/8″ Pl.	0.361	49 900	71 200	16.30	30.80	60 400	67 900	66 570	64 370	93.50	90.42	0.16	0.90	0.087
1b	"	"	"	"	"	"	61 000	68 300	67 240	64 750	94 44	90.95	"	"	"
2a	12″ × 1/2″ Pl.	0.369	55 100	74 100	13.20	21.10	64 200	71 800	69 230	66 610	93.45	89.90	0.21	0.65	0.091
2b	"	"	"	"	"	"	64 000	71 500	69 010	66 330	93.14	89.53	"	"	"
3a	4″ × 4″ × 7/16″ L.	0.435	48 200	82 200	8.10	15.10	80 300	87 100	73 460	68 530	89.36	83.39	0.26	0.53	0.115
3b	"	"	"	"	"	"	81 000	87 500	74 090	68 840	90.13	83.75	"	"	"
4a	6″ × 1/2″ Pl.	0.515	43 400	70 800	18.80	28.90	70 900	78 300	54 780	52 040	77.37	73.50	0.18	0.57	0.091
4b	"	"	"	"	"	"	73 000	80 100	56 400	53 240	79.66	75.20	"	"	"
5a	3½″ × 3½″ × ½″ L.	0.505	46 300	76 400	18.10	29.50	75 300	82 900	59 340	56 190	77.67	73.53	0.15	0.51	0.125
5b	"	"	"	"	"	"	77 000	84 500	60 670	57 280	79.41	74.98	"	"	"
6a	15″ × ½″ Pl.	0.518	58 000	79 900	10.30	16.30	84 900	94 700	65 210	62 570	81.61	78.31	0.17	0.41	0.145
6b	"	"	"	"	"	"	81 000	95 000	62 220	62 770	77.87	78.57	"	"	"
7	4″ × 3/8″ Pl.	0.390	.....	125 800	in 2″ = 7.50 " 3″ = 6.67 " 4″ = 6.25 " 5″ = 6.00 " 6″ = 7.50	4.41	87 000	93 000	88 760	86 020	70 56	68.37	0.71	0.14	0.079

TABLE E1.

Weak and Rotten Steel.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
				Tensile Test.				Punching Test.		Results Obtained by Calculation.					Analyses.			
Specimen Number.	Remarks on Quality.	Description cut from—	Thickness of Specimen.	Elastic limit per square inch.	Ultimate strength per square inch.	Elongation in 8 ins. Percent.	Reduction of area. Percent.	Pressure applied in pounds to punch with $\frac{3}{4}$-in. punch.	Pressure applied in pounds to punch with $\frac{7}{8}$-in. punch.	$\frac{P}{c \times t}$ with $\frac{3}{4}$-in. diameter punch.	$\frac{P}{c \times t}$ with $\frac{7}{8}$-in. diameter punch.	Percent. of $\frac{P}{c \times t}$ with $\frac{3}{4}$-in. diameter punch to ultimate strength.	Percent. of $\frac{P}{c \times t}$ with $\frac{7}{8}$-in. diameter punch to ultimate strength.	Carbon. Per cent.	Manganese. Per cent.	Phosphorus. Per cent.	Sulphur. Per cent.	Silicon. Per cent.
1a.	Poorly made—rotten.	2 x 3 x $\frac{5}{16}$" L.	0.313"	30 100	47 800	19.80	38.50	30 100	34 400	38 270	37 615	80.06	78 49	0.12	0.31	0.095	0.078	0.094
1b.	"	"	"	"	"	"	"	29 700	33 500	37 760	36 610	79 00	76.65	"	"	"	"	"
2a.	Badly burned.	3 x 3 x $\frac{3}{8}$" L.	0.390"	35 400	61 000	8.50	12.20	37 400	42 000	38 160	36 860	62 56	60.45	0.10	0.14	0.072	0.035	0.001
2b.	"	"	"	"	"	"	"	36 700	42 500	37 440	37 300	61.34	61.14	"	"	"	"	"
3a.	Poorly made.	3 x 3 x $\frac{3}{8}$" L.	0.379"	32 500	48 900	17.50	33.10	39 900	44 700	41 830	41 650	85.68	85.18	0.14	0.377	0.101	0.071	0.087
3b.	"	"	"	"	"	"	"	40 300	45 500	42 410	42 400	86.52	86.70	"	"	"	"	"
4a.	Weak steel.	16 x $\frac{7}{16}$" Pl.	0.445"	30 900	48 300	16.30	31 20	45 500	50 200	40 680	38 610	84.22	79 94	0.087	0.229	0.041	0.065	0.022
4b.	"	"	"	"	"	"	"	44 900	50 000	40 140	38 450	83 11	79.60	"	"	"	"	"
5a.	"	20 x $\frac{1}{2}$" Pl.	0.523"	32 200	46 000	18.90	31.90	50 800	56 400	38 650	36 920	84 02	80.26	0.067	0.12	0.065	0.032	0.037
5b.	"	"	"	"	"	"	"	50 200	56 100	38 190	36 720	83.02	79.83	"	"	"	"	"
6a.	Not sufficiently worked.	10 x $\frac{3}{4}$" Pl.	0.770"	37 400	58 800	15.25	36.50	62 000	72 000	32 040	32 010	54.49	54.43	0.16	0.29	0.055	0.042	0.008
6b.	"	"	"	"	"	"	"	64 000	71 000	33 080	31 570	56 26	53.69	"	"	"	"	"
7a.	"	12 x $\frac{9}{16}$" Pl.	0 900"	39 800	64 300	17.10	28 20	.....	74 000	.....	28 150	.....	43.78	0.19	0.34	0.063	0.033	0.010
7b.	"	"	"	"	"	"	"	.....	75 000	.....	28 530	.....	44.37	"	"	"	"	"
8.	Burned.	7 x $\frac{13}{16}$" Pl.	0.810"	.....	.....	.....	.....	90 000	.....	44 210	.....	.....	.....	.....	.....	.....	.....	.....

TABLE E2.

Tests of Burned Steel.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
		Pressure Required to Punch Normal Specimens of Steel.							Pressure to Punch Same Specimens of Steel After Being Burned.				Variance in Pressure With Burned Steel From That of the Same Steel in the Normal State.			
Specimen Number.	Description cut from—	Actual pressure, $\frac{1}{4}$-in. punch P.	Actual pressure, $\frac{1}{4}$-in. punch P.	Pressure per square inch, $\frac{3}{4}$-in punch $\frac{P}{c \times t}$.	Pressure per square inch, $\frac{1}{4}$-in. punch $\frac{P}{c \times t}$.	Percent. $\frac{P}{c \times t}$ to ultimate strength. $\frac{3}{4}$-in. punch.	Percent. $\frac{P}{c \times t}$ to ultimate strength. $\frac{1}{4}$-in. punch.	Thickness of burned specimen.	Actual pressure in pounds. $\frac{3}{4}$-in. punch. P.	Actual pressure in pounds. $\frac{1}{4}$-in. punch. P.	Pressure per square inch. $\frac{1}{4}$-in. punch. $\frac{P}{c \times t}$.	Pressure per square inch. $\frac{1}{4}$-in. punch. $\frac{P}{c \times t}$.	Pounds variance. $\frac{1}{4}$-in. punch.	Percent. with $\frac{1}{4}$-in. punch of U.	Pounds variance with $\frac{1}{4}$-in. punch.	Percent. variance. $\frac{1}{4}$-in. punch of U.
1 a	6½" x $\frac{3}{16}$" Pl.	21 600	25 200	46 450	46 610	70.68	70.92	0.105	12 300	11 000	46 610	35 860	+ 160	70.92	−10 750	54.56
b	"			"	"	"	"	"	11 500		43 580		− 2 870	66.30		
2 a	3" x 3" x ¼" L.	28 300	31 800	45 970	50 320	74.64	72.14	0.145	16 900	17 800	46 070	42 020	+ 2 900	74.80	− 8 300	68.23
b	"		32 200	45 970	50 320	"	73.04	"	16 800	16 000	45 790	37 770	− 180	74.34	−12 550	61.33
3 a	3" x 3" x $\frac{5}{16}$" L.	37 000	41 500	47 500	45 830	77.90	75.16	0.210	24 600	27 200	45 470	44 320	− 2 030	74.58	− 1 510	72.70
b	"	36 800	41 500	47 240	45 830	77.48	"	"	23 900	27 900	45 280	45 470	− 1 960	74.28	− 360	74.58
4 a	10½" x $\frac{3}{16}$" Pl.	45 700	46 800	49 810	43 880	86.32	"	0.270	32 800	37 000	48 340	46 900	− 1 470	82.82	+ 3 020	80.35
b	"	44 900	48 900	47 960	45 850	82.16	78.54	"	33 700	36 600	49 660	46 270	+ 1 700	85.08	+ 420	79.27
5 a	25" x $\frac{7}{16}$" Pl.	51 200	58 300	47 370	46 410	76.55	74.98	0.365	42 100	46 100	45 880	43 280	− 1 440	74.09	− 3 130	69.91
b	"	51 600	58 400	47 730	46 490	77.10	78.09	"	41 100	44 700	44 800	41 970	− 2 930	72.36	− 4 520	67.79
6 a	20" x ½" Pl.	60 300	66 900	47 980	45 700	77.27	73.88	0.430	51 500	58 000	47 640	46 180	− 340	76.86	+ 440	74.50
b	"	59 400	68 100	47 270	45 240	76.26	72.99	"	50 000	55 700	46 250	44 340	− 1 020	74.61	− 900	71 53
7 a	6" x 6" x $\frac{5}{16}$" L.	70 000	79 000	48 020	46 620	76.98	74.74	0.480	58 800	68 400	48 750	48 780	+ 730	78.15	+ 2 160	78.20
b	"	69 000	78 300	47 340	46 210	75.89	74.08	"	55 000	64 300	45 600	45 860	− 1 740	73.10	− 350	73.51
8 a	13" x $\frac{5}{16}$" Pl.	61 000	71 400	43 340	43 640	68.48	68.95	0.475	52 000	60 400	43 550	43 520	+ 210	68.81	− 120	68.76
b	"	62 000	71 500	44 050	43 710	69.60	69.06	"	53 200	57 400	44 550	41 350	+ 500	70.39	− 2 360	65.33
9 a	5" x 5" x ⅜" L.	74 000	83 300	49 070	47 620	78.56	76.18	0.525	63 000	70 700	47 760	46 090	− 1 310	76.56	− 1 430	73.81
b	"	74 300	83 200	49 270	47 460	78.98	76.10	"	62 200	70 100	47 160	45 700	− 2 110	75.60	− 1 760	73.26
10 a	6" x 6" x $\frac{13}{16}$" L.	86 000	93 100	61 070	47 570	84.58	77.89	0.600	74 100	84 100	49 140	47 970	− 1 930	80.43	+ 400	78.52
b	"	85 500	93 700	60 830	47 870	83.20	78.34	"	71 100	83 100	47 150	47 400	− 3 680	77.16	− 470	77.59
11 a	6" x 6" x $\frac{9}{16}$" L.	67 100	76 400	48 100	47 110	78.97	77.34	0.460	54 500	67 700	47 140	50 370	− 960	77.39	+ 3 260	82.70
b	"	65 700	76 100	47 090	46 930	77.32	77.06	"	56 700	63 500	49 050	47 250	+ 1 960	80.52	+ 320	77.57
12 a	4" x 4" x ¼" L.	82 000	97 300	45 010	45 930	74.82	76.34	0.685	70 200	83 800	40 770	41 880	− 4 240	67.77	− 4 050	69.62
b	"	83 800	96 700	46 050	45 650	71.46	75.88	"	70 000	80 100	40 650	40 630	− 5 400	67.57	− 5 620	66.54
c	"							"	68 100		39 550			65.74		
13 a	10" x ¼" Pl.	83 000	92 000	43 740	41 670	74.44	69.00	0.675	79 600	89 200	46 930	45 240	+ 2 190	77.72	+ 3 570	74.91
b	"	85 000	99 000	44 000	44 840	72.24	74.26	"	70 100	88 400	41 330	44 820	− 3 470	68.44	− 10	74.23
c	"							"	73 900		43 570			72 15		
14 a	10" x $\frac{13}{16}$" Pl.	90 300	97 600	43 980	40 850			0.735	80 900	92 200	43 800	42 940	− 180		+ 2 090	
b	"	88 100	100 400	42 920	42 960			"	78 000	88 200	42 230	41 080	− 690		− 980	
15 a	24" x ¼" Pl.	101 000	114 000	45 640	44 340	78.86	76.62	0.810	91 300	104 000	44 840	43 960	− 800	77.48		79.42
b	"		107 000		41 630		71.94	"		99 100		41 880			− 2 460	72.37
c	"							"		106 100		44 800			+ 3 170	77.41
16 a	6" x 6" x $\frac{15}{16}$" L.	117 000	120 000	49 630	43 860	79.92	70.54	0.720	88 100	99 300	48 670	47 200	− 960	78.37	+ 3 400	76.00
b	"	110 000	120 000	46 600	"	75.14	"	"		96 500		45 860			+ 2 060	73.84
17 a	⅝" x 6" x $\frac{15}{16}$" L.		119 000		44 760			0.840	95 100		45 050					
b	"							"	93 000		44 650					

TABLE F.

Steel Over $\frac{5}{8}$-In. Thickness.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
			Tensile Test.				Punching Test.		Results Obtained by Calculation.				Analyses.		
Specimen Number.	Description cut from —	Thickness of specimen.	Elastic limit per square inch.	Ultimate strength per square inch.	Elongation in 8 ins. Percent.	Reduction of area. Percent.	Pressure applied in pounds to punch with $\frac{3}{4}$-in. punch.	Pressure applied in pounds to punch with $\frac{7}{8}$-in. punch.	$\frac{P}{c \times t}$ with $\frac{3}{4}$-in. diameter punch.	$\frac{P}{c \times t}$ with $\frac{7}{8}$-in. diameter punch.	Percent. of $\frac{P}{c \times t}$ with $\frac{3}{4}$-in. diameter punch to ultimate strength.	Per cent. of $\frac{P}{c \times t}$ with $\frac{7}{8}$-in. diameter punch to ultimate strength.	Carbon. Percent.	Manganese. Percent.	Phosphorus. Percent.
1.....	4 x 4 x $\frac{13}{16}$" L.	0.690"	40 000	61 300	24.25	48.50	80 100	89 100	46 200	44 200	75.38	72.12	0.15	0.35	0.075
2.....	" "	0.683"	42 500	64 800	23.75	45.80	84 200	94 000	49 060	48 960	75.70	75.56	0.17	0.42	0.072
3.....	12 x $\frac{11}{16}$" Pl.	0.688"	41 300	62 600	25.30	51.70	81 300	92 100	47 030	45 820	75.25	73.30	0.24	0.41	0.011
4.....	10 x $\frac{5}{8}$" Pl.	0.755"	37 100	58 200	25.70	53.30	82 100	92 300	43 260	41 850	74.32	71.90	0.13	0.58	0.060
5.....	" "	0.764"	36 300	56 100	27.10	57.25	80 400	88 700	41 860	39 710	74.62	70.78	0.20	0.39	0.007
6.....	10 x $\frac{13}{16}$" Pl.	0.817"	39 300	59 800	29.25	59.30	90 300	97 500	43 960	40 850	73.51	68.32	0.13	0.53	0.077
7.....	" "	0.820"	40 700	64 500	24.80	50.70	92 100	101 400	44 690	41 900	69.29	64.96	0.13	0.51	0.073
8.....	24 x $\frac{7}{16}$" Pl.	0.870"	39 500	60 400	25.80	57.75	101 000	114 000	46 180	44 820	76.46	74.19	0.26	0.51	0.009
9.....	20 x $\frac{7}{8}$" Pl.	0.878"	42 000	61 600	24.90	49.50	98 300	107 200	44 540	41 800	72.30	67.86	0.14	0.49	0.035
10.....	6 x $\frac{5}{8}$" bar.	0.882"	44 000	65 900	23.25	45.80	99 900	106 500	45 060	41 830	68.45	62.73	0.15	0.44	0.085
11.....	6 x 6 x $\frac{15}{16}$" L.	0.910"	40 100	62 200	23.00	44.50	97 500	105 700	42 630	39 750	68.56	63.91	0.12	0.57	0.094
12.....	" "	0.920"	35 300	54 500	25.10	47.70	97 100	104 200	41 960	38 770	77.00	71.14	0.12	0.34	0.043
13.....	21 x 1" Pl.	1.010"	39 200	59 300	24.60	48.80	.....	121 000	.....	41 000	.....	69.14	0.11	0.58	0.092
14.....	8 x 1" bar.	1.008"	37 500	62 400	25.10	49.90	.....	120 400	.....	40 880	.....	65.62	0.22	0.47	0.018

TABLE G.

Action of Speed of Punching in Modifying the Results.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
				Punching Tests—$\frac{3}{4}$-In. Diameter Punch.							Punching Tests—$\frac{7}{8}$-In. Diameter Punch.						
Specimen Number.	Remarks on quality.	Description cut from—	Thickness of specimen.	Pressure applied in pounds to punch in 10 seconds.	Pressure applied in pounds to punch in 30 seconds.	Pressure applied in pounds to punch in 60 seconds.	Pressure applied in pounds to punch in 2 minutes.	Pressure applied in pounds to punch in 3 minutes.	Pressure applied in pounds to punch in 5 minutes.	Pressure applied in pounds to punch in 10 minutes.	Pressure applied in pounds to punch in 10 seconds.	Pressure applied in pounds to punch in 30 seconds.	Pressure applied in pounds to punch in 60 seconds.	Pressure applied in pounds to punch in 2 minutes.	Pressure applied in pounds to punch in 3 minutes.	Pressure applied in pounds to punch in 5 minutes	Pressure applied in pounds to punch in 10 minutes.
1.....	Good steel..	5 x 3½ x ⅜" L.	0.423"	47 700	47 900	48 000	48 100	48 000	48 100	48 100	52 400	52 600	52 800	52 800	52 900	52 900	52 800
2.....	" ..	10 x ⅜" Pl.	0.365"	41 000	41 100	41 200	41 200	41 200	41 100	41 400	45 700	46 800	46 700	46 800	46 900	47 000	46 800
3.....	Hard steel..	"	0.367"	51 800	51 800	52 000	52 100	52 000	52 100	52 100	58 400	58 100	58 200	58 300	58 400	58 500	58 500
4.....	Good steel..	"	0.362"	46 300	47 100	47 200	47 200	47 100	47 700	47 500	53 300	53 900	53 800	53 500	53 900	54 000	53 600
5.....	" ..	3 x 3 x ⅜" L.	0.405"	46 800	47 500	46 900	47 000	47 400	47 500	47 200	54 200	54 100	54 300	54 400	54 400	54 400	54 500
6.....	" ..	5 x 3 x ⅜" L.	0.370"	45 300	45 400	45 500	45 700	45 700	45 700	45 700	52 100	52 300	52 300	52 500	52 600	52 500	52 500
7.....	" ..	4 x 4 x ⅜" L.	0.366"	43 000	43 400	43 500	43 400	43 500	43 500	43 500	50 200	50 100	50 200	50 400	50 600	50 300	50 400
8.....	Hard steel..	15 x 1" Pl.	0.256"	36 300	36 300	36 400	36 500	36 400	36 600	36 500	39 700	39 900	39 900	40 100	40 200	40 100	40 100
9.....	Good steel..	16 x ½" Pl.	0.505"	55 000	56 100	56 400	56 500	56 200	56 300	56 400	64 200	64 300	64 300	64 400	64 600	64 600	64 500
10.....	" ..	5 x 5 x ⅜" L.	0.610"	74 000	74 300	74 800	75 000	75 200	75 500	75 600	83 200	83 300	83 900	83 800	84 000	84 100	84 000
11.....	" ..	6 x 6 x 1⅛" L.	0.680"	85 500	85 600	85 900	86 000	86 200	86 400	86 500	90 300	90 700	91 200	91 900	91 700	91 800	91 700
12.....	" ..	4 x 4 x ¼" L.	0.770"	82 000	82 800	83 500	83 700	83 800	83 800	83 700	96 700	96 800	97 000	97 100	97 300	97 200	97 300

TABLE H.

Variations Due to Location of Punched Holes and Their Environment.

1	2	3	4	5a	5b	6a	6b	7a	7b	8a	8b	9a	9b	10a	10b	11a	11b	12a	12b	13a	13b	14a	14b	15a	15b	16a	16b	17a	17b
Specimen Number.	Remarks on Quality.	Description cut from—	Thickness of Specimen.	Pressure applied in pounds to punch hole whose center is 3 in. from edge of plate.		Pressure applied in pounds to punch hole whose center is 2 in. from edge of plate.		Pressure applied in pounds to punch hole whose center is 1½ in. from edge of plate.		Pressure applied in pounds to punch hole whose center is 1½ in. from rolled edge and 1¼ in. from center of adjoining similarly punched hole.		Pressure applied in pounds to punch hole whose center is 1 in. from rolled edge of plate.		Pressure applied in pounds to punch hole whose center is 1 in. from sheared edge of plate.		Pressure applied in pounds to punch hole whose center is ¾ in. from rolled edge of plate.		Pressure applied in pounds to punch hole whose center is ¾ in. from rolled edge.		Pressure applied in pounds to punch hole whose center is ⅝ in. from rolled edge.		Pressure applied in pounds to punch hole whose center is ⅝ in. from sheared edge.		Pressure applied in pounds to punch hole whose center is ½ in. from rolled edge.		Pressure applied in pounds to punch hole whose center is ½ in. from sheared edge.		Pressure applied in pounds to punch hole whose center is ⅜ in. from planed edge.	
				¼" Punch.	⅜" Punch.	¼" Punch.	⅜" Punch.	¼" Punch.	⅜" Punch	¼" Punch.	⅜" Punch.	¼" Punch.	⅜" Punch.	¼" Punch.	⅜" Punch.	¼" Punch.	⅜" Punch.	¼" Punch.	⅜" Punch	⅛" Punch.	¼" Punch.	¼" Punch.	⅜" Punch.	" Punch.	⅜" Punch	¼" Punch.	⅜" Punch.	¼" Punch.	⅜" Punch.
1...	Hard steel.	4 x ⅜" bar.	0.390"	87 000	98 000	86 500	99 000	87 500	97 800	87 500	96 200	86 000	96 000	84 000	92 000	83 000	92 000	80 000	87 000	79 000	87 900	74 000	76 000	71 200	73 000	69 100	68 000	70 000	71 500
2...	Good steel.	7 x 1⅛" bar.	0.810"	90 000	.....	.....	.....	89 800	.....	89 500	.....	90 000	.....	86 800	.....	88 000	.....	85 000	.....	83 000	.....	79 000	.....	79 200	.....	73 000	.....	82 200	.....
3...	"	6 x 3½ x ⅜" L.	0.460"	.....	.....	.....	61 500	.....	61 500	.....	60 300	.....	61 200	.....	60 300	.....	58 000	.....	57 300	.....	57 100	.....	55 200	.....	55 100	.....	52 000	.....	55 400
4...	"	24 x 7/16" Pl.	0.435"	.....	56 200	.....	56 300	.....	56 300	.....	56 000	.....	56 000	.....	55 000	.....	55 600	.....	54 500	.....	54 100	.....	53 700	.....	53 600	.....	52 000	.....	52 800
5...	"	6½ x 9/16" Pl.	0.562"	67 500	75 600	67 700	76 200	67 100	75 800	66 900	75 200	66 100	74 200	65 000	74 000	65 100	74 500	64 700	73 800	65 100	73 900	64 300	72 500	63 800	72 900	62 000	71 400	63 900	73 100
6...	"	10 x ⅜" Pl.	0.365"	44 200	.....	44 000	.....	43 700	.....	43 200	.....	43 200	.....	42 700	.....	43 100	.....	42 100	.....	41 500	.....	40 600	.....	40 700	.....	40 100	.....	40 900	.....
7...	"	10½ x ⅜" Pl.	0.365"	45 700	59 500	45 500	51 000	45 200	50 700	45 100	50 900	45 100	50 100	44 700	50 000	44 200	49 800	43 700	49 500	44 000	49 100	43 600	48 600	44 100	48 200	43 700	48 100	44 400	49 000
8...	"	3 x 3 x ¼" L.	0.210"	24 000	27 200	24 100	27 400	24 300	27 500	24 200	27 100	24 500	27 700	23 200	26 500	23 900	27 900	23 100	26 300	23 700	26 500	23 000	25 800	23 100	25 900	22 700	25 000	23 000	26 100
9...	Burned steel.	3 x 3 x 5/16" L.	0.145"	16 900	17 800	17 000	18 500	17 200	18 100	17 100	18 000	16 700	18 000	16 000	17 500	16 200	17 900	15 800	17 000	15 900	17 100	15 300	16 100	15 200	16 200	14 800	16 000	15 000	16 400
10...	Good steel.	20 x ½" Pl.	0.510"	60 300	66 900	60 300	66 800	60 100	66 800	60 000	67 100	59 800	66 600	59 000	66 200	57 900	66 000	57 700	65 400	57 800	65 500	57 100	65 000	58 200	64 400	56 100	64 000	56 500	64 700

Emery testing machine, and the remarkable uniformity of the results is shown in Table G. In a general way, the pressure applied rapidly, as in 10 seconds, gives slightly lower results than when the same conditions are varied only by the time being considerably longer in applying the pressure upon the punch. The entire difference, however, is very slight, comparatively, and will not be a cause of error in this method of testing steel by any ordinary variances due to speed.

Ten seconds is about as short a time as it has been found practicable to make a punching and record its results with the apparatus at the writer's command.

APPENDIX H.

Variations in Power Required in Punching Due to Location of Punched Holes and their Environment.

Both $\frac{3}{4}$ and $\frac{7}{8}$-in. holes were punched in varying locations in steel specimens. The results are given in Table H, which is self-explanatory.

Holes punched near the sheared edge require from 10 to 20% less work than those punched nearer the center of the plate or angle, the loss being greater with hard steel than with soft. Slight differences in location of the center of the punch do not seem to make great differences, not over at most 1 000 lbs. or so, in the results.

As a rule, plates tested by this method show slightly higher shearing strength than angles or other shapes of the same steel. The writer's experience is that both the tensile and the shearing strength of universal mill-plates increases by from 5 to in some cases 10%, as the test specimens are selected towards the center of the plates, from the results obtained in test specimens selected near the edge of the plate. This fact is made use of at the mills in the selection of tensile test specimens to fill varying requirements.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

642.

(Vol. XXX.—October, 1893.)

THE USE OF MILD STEEL FOR ENGINEERING STRUCTURES.

By GEORGE C. MEHRTENS, Regierungs- und Baurath, Member of Royal Board of Railway Directors at Bromberg, Prussia.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

The importance of mild steel as material for use in engineering structures, and its probable adoption in the construction of bridges, was noted by the author in an article published in 1882.* The distrust in such use will surely disappear more rapidly than did the prejudice against puddled and rolled iron as opposed to that fined in the hearth and forged under the hammer.

More than 10 years have passed since, and in consequence of the general adoption of the inventions of Martin and Thomas, there has been an enormous increase in the production and use of low steel; and

* Notice respecting fabrication of iron and iron bridges, "Deutsche Bauzeitung," 1882.

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

in the choice of material for important structures, wrought iron and mild steel are now considered equivalent, although a few prefer the former, and that mostly from ignorance.

In face of the numerous engineering structures now existing which have been built of mild steel, there cannot be any well-founded objection to its use. There are, however, in this much-debated field many questions yet awaiting solution, and he who will conscientiously decide them has much labor before him. To aid those who are still undecided, those who favor the use of mild steel should agree and do their part in spreading a knowledge of its qualities and advantages. The author has been for many years such an advocate and takes pleasure in embracing this opportunity to make his opinions known.

Mild steel up to the year 1860 had not been used for engineering structures, neither had the Bessemer process come into general use before that time—the only materials available were weld steel and cast steel.*

Cast steel is often called ingot steel (*fluss-stahl*) but this is not correct. The term ingot steel to-day includes generally only those kinds produced directly by a single melt from the raw materials. Cast steel is made by several meltings from the ingot metal, and, therefore, does not belong to them. The beginning of the use of ingot steel for structures was contemporaneous with the development of the Bessemer process, and one of its first applications was to the building of merchant ships in England. The first of these was the *Jason*, an English ship of 452 tons burden and destined to ply in the Black Sea. Five channel steamers of the London, Chatham and Dover Railway were built in 1860 and 1861. The next step was the use of Bessemer steel in building boilers in men-of-war and railway locomotives in France and America in 1861 to 1864; also, in England the manufacture from the same material in 1865 of driving shafts for marine engines. It was used in bridges of the Dutch State railways in 1863–64. Owing to the irregular quality and occasional brittleness of Bessemer steel, adoption

* It is worthy of note that the first adoption of weld steel in bridge-building (1818) was by Mitis for the Charles chain bridge over the Danube canal at Vienna. The steel used for railways, war material, and especially for rails, tires, axles, guns, etc., was crucible cast steel. At the Exposition in London, in the year 1851, the far-famed Krupp works at Essen made the first exhibition of heavy cast-steel ingots and barrels of heavy guns; at that Exposition also was to be seen German puddled steel of the work of Lohage, at Haspe, in Westphalia. At the Paris Exposition, in 1855, was exhibited the first cast-steel boiler from the French works of Petin, Gaudet & Co.

of it for ship-building made slow progress, as these made its manipulation very difficult.

The extensive steel works of Terre Noire, in France, prominent in this department of metallurgy, made every effort to produce a mild ingot steel. In the years 1868 and 1869 the Transatlantic Company supplied a mild ingot steel, 64 000 to 71 000 lbs. tenacity per square inch (45 to 50 kg. per square millimeter), and 18 to 26% elongation on a testing piece 4 ins. long. The authorities of the French railways decided that the material was not suitable for locomotive boilers and preferred an ingot steel of at least 78 000 lbs. tenacity per square inch (55 kg. per square millimeter). Plates of 68 000 to 75 000 lbs. per square inch (48 to 53 kg. per square millimeter), with 20 to 24% elongation, were often refused as being unsuitable.

In 1870 the Terre Noire works offered to the French navy to supply plates and angles of soft Bessemer steel to take the place of wrought-iron plates and angles; they had then succeeded, by the plentiful use of ferro-manganese, in producing satisfactory plates. The Navy Department at first refused to accept such plates, but finally allowed their use in ship-building. In 1874 the first French man-of-war named the *Redoutable* was built of mild steel. She was a first-class armor-plated frigate 302 ft. long and 64 ft. beam. These works, and those of Creuzot, together, furnished the plates and bars, which were manufactured partly by the Bessemer process and partly by the Martin open-hearth process. The requirements were a tenacity between 64 000 to 68 000 lbs. (45 to 48 kg. per square millimeter), and of 18 to 22% elongation. The successful use of mild steel for ships by the French decided their engineers upon building marine boilers of the same material, with the exception of the fire-plates, and had a great influence on the adoption of the material by English engineers for ships and boilers.*

The Pennsylvania Railroad Company first introduced Bessemer steel in America in locomotive boilers in the year 1863, but the steel was too hard and brittle for such use, and this road was deterred from making further experiments in that direction.

---

* In the beginning of 1878 Mr. Parker, Chief Engineer of Lloyd's, stated that there was then but one English steamer in possession of a steel boiler; a year later there were 120, and in 1881 about 1 100 steel boilers weighing 1 700 tons total. In 1878 there were but five large English steamers built of steel, while in 1883 there were 116 building.

As the material came more largely into use, complaints against the irregularity of the material became more frequent, the principal difficulty being its hardness as well as its irregularity; and not only that, but those using the metal had not yet learned its proper treatment, and the plates were too thin. The first plates made for steel boilers had a tenacity of 85 000 to 92 000 lbs. (60 to 65 kg.), and an elongation of but 7 to 10%; and it was supposed that with this much greater strength it would be possible to materially diminish the thickness of the plates below those made of wrought iron. Soon after the World's Exposition in 1855, where the first cast-steel boiler appeared, the French authorities allowed the makers to diminish the thickness of the plates one-third from that which had been required in wrought iron since the year 1843. In 1861 the reduction in thickness for steel having a tenacity of 85 000 lbs. (60 kg.) and 6.6% elongation was one-half from that of wrought iron; but the results were not favorable, and the steel works were soon forced to offer a material of less tenacity and more ductility. The requirements were therefore reduced to a tenacity of 78 000 lbs. (55 kg.) or less, and the elongation was increased to 15% or more. Even with this, between the years 1870 and 1880, many explosions occurred and many careful examinations were made to determine their cause. It was found on examining the rivet holes that there were incipient changes in the metal, many cracks around them, and points near them were corroded with rust, all caused by the shock of tools in manufacturing. It was evident that the material was unsuitable, and that the treatment must be changed. The use of Bessemer steel in bridge-building was very early tried, as was previously mentioned. First on the Dutch State railways in 1863–64, then in England and Austria. Following the use of steel in the bridges of the Dutch railways, in 1874 a bridge was built of Bessemer steel, crossing over the railway at the station of Pesth, by H. Schmidt, engineer. Several superstructures of bridges on the branch line of Ebersdorf-Würbenthal, have been built since 1881.

The first use of cast steel for bridges was in America, for the St. Louis Arch Bridge and for the wire of the East River Bridge. These gave an impetus to the use of ingot metal, and before 1880 the Glasgow and Plattsmouth bridges over the Missouri River were also built of ingot metal. For the superstructure for the bridge in Glasgow, steel eye-bars were applied for the first time.*

* *Transactions* Am. Soc. C. E., October, 1892.

While in the 10 years succeeding 1860, Bessemer metal was most widely used, on account of bad failures in its use in ships, the open-hearth metal came slowly to be preferred by engineers. From 1870 to 1880, experiments were made in Europe in building bridges and other structures of Martin open-hearth metal; these were induced doubtless by the unsatisfactory results of the introduction of Bessemer metal in the large span bridges of the Dutch State railways. The invention of Thomas in 1878, and its use with the Martin furnace in 1882, removed the distrust and prejudice then existing.

Since 1880 the introduction of mild steel in all kinds of engineering structures has steadily increased.

Table No. 1 gives a connected statement of the most remarkable mild-steel bridges built in Europe in 1880 to 1890. This excludes those in acid Bessemer metal.

The writer was unable to make this complete for American bridges because distinction is not there usually made in the description as to whether the steel employed was Bessemer or Martin steel, and also because some of the information sought for by American bridge works was not obtained.

In the table the countries of Central Europe including France are mostly represented; this is by reason of the dominance in England and America of the acid Bessemer metal up to the year 1880. In the first design of the Forth Bridge, 1881–82, this metal was provided, and in America, as stated, there were in 1880 but two bridges of this material. English publications respecting steel bridges, as also those of America, do not name the kind of steel used, although English steel works furnished several bridge superstructures in mild steel for foreign countries, among them the well-known Sukkur Bridge for the India North West Railway; the Chenal Bridge, for the India State Railway, 17 spans of 206 ft. each. In England also a remarkable railway swing bridge over the Dee, designed by Mr. Fox, 2 truss spans each 120 ft. long, and a swing bridge having unequal arms of 140 and 86 ft. each, and 727 tons weight. The links of the Hammersmith Suspension Bridge over the Thames were of mild steel.

The use of basic mild steel for bridges increased most rapidly in those countries in Central Europe which first adopted it, especially Germany, Austria and France, while the use of acid Martin metal predominated in England and America. So far as is known the first mild-

steel bridges were erected in the years 1880–82, at Königsberg, in Prussia, by City Engineer Frühling. These were draw-bridges, and the object in using steel was to lighten them as much as possible. In 1885, Harkort, at Duisburg, introduced Thomas metal for a large bridge to be erected in Sumatra. Basic Martin steel bridges were erected in Austria and France in 1886–87, and in 1881* the great Bohemian works at Kladno in Austria-Hungary first produced plates and bars by the Thomas process, while the first Bohemian mild-steel bridge, which was a bridge with a single bowstring girder span 158 ft. long, over the Elbe at Nemcic, was still built of acid Bessemer metal. After the year 1887, the table names several Bohemian road-bridges built of Thomas and Martin mild steel, also railroad bridges.

A committee of the Austrian Society of Engineers and Architects made exhaustive experiments with satisfactory results on Martin open-hearth steel, after which it was adopted on the Austrian State railways.† It was probably these results which induced a promulgation of a ministerial order on January 29th, 1892, excluding Thomas mild steel entirely from use in bridges built on these roads. Later experiments,‡ however, with both kinds of metal, by the Government at Prague, seemed to give equally good results with each.

There are natural reasons why up to the present time the acid Martin metal has been preferred to Thomas metal for use in structures. The Martin process originated 13 years earlier than the invention of the dephosphorizing process in the Bessemer converter, and had therefore begun to be widely used, and it was difficult for the Thomas metal to make headway. If recently the Thomas metal has been more favorably looked upon, it will doubtless be due to the favorable results of comparative tests made by the Prussian State Railway Board in the year 1889, and since that date, in connection with the erection of the large Weichsel bridges near Dirschau, Marienburg and Fordon. These results have been published in several technical papers.§

The Prussian State Railway Department intended to use mild steel for the superstructures of the new bridges at Dirschau and Marienburg; but it was not thought desirable, not because of distrust of the material, but on account of the possible delays. There was at the time

* Tetmayer. A contribution for the mild-steel question, Schweizer Bauzeitung, 1892.

† "Allgemeine Bauzeitung," 1891, Part 2.

‡ Published by Professor Steiner at Prague, "Zeitschrift des Oesterreichischen Ingenieur und Architekten Vereines," 1892, Nos. 8 and 10.

§ "Stahl und Eisen," 1891, Nos. 8 and 9. "Centralblatt der Bauverwaltung," 1891, p. 395.

an abundant supply of mild steel of proper quality available, but the necessary testing and inspection of such large quantities of material, 7 000 tons for the Weichsel Bridge and 1 500 for the Nogat Bridge, could not be completed in the time stipulated. For these reasons the use of low steel was limited to some of the highly strained members in the two bridges, and it was decided to construct a smaller bridge over the Wallgraben at Marienburg (No. 17, Table No. 1) as a preliminary test, using low steel throughout. Martin steel alone was provided for in the bridge, because at the time of drawing the specifications Thomas steel was not thought sufficiently reliable, and yet it was impossible to resist the conclusion that Thomas metal could have been used for many purposes without scruple.

The results of the tests mentioned above were so favorable for the Thomas metal that this material was admitted on a large scale, together with Martin mild steel, in the construction of the Fordon Bridge over the Weichsel.

Up to the beginning of the present year tests have been made from about 1 200 blows containing nearly 10 000 tons of finished products. These have been most carefully made by tension, bending, and drop tests. Bending tests have been made after subjecting the specimens to a very low temperature. Results of the tests are shown in Plates I and II; they represent 700 blows for the Thomas and 500 for the Martin steel. Not a single blow of the Thomas steel failed, but 16 blows of the Martin steel were not satisfactory.*

The Royal Board of Directors in Bromberg, and the writer, have been in receipt of many inquiries from German and foreign departments respecting these tests, and it is evident that the successful use of basic steel in the prominent bridges over the Weichsel has given great impetus to its general adoption for bridge structures.

Table No. 2 gives a concise summary of the most remarkable mild-steel bridges erected during the years 1890–93 in Europe. Many small bridges have been omitted from this table. Acid metal has been excluded because it is now used but very little. †

* "Stahl und Eisen," 1892, No. 13, and 1893, No. 7. "Zeitschrift des Vereines Deutscher Ingenieure," 1892, Vol. 36, p. 778. "Centralblatt der Bauverwaltung," 1892, pp. 68, 83 and 285. "Le Genie Civil," 1892, p. 5–11. "The Engineering and Mining Journal," 1891, December, pp. 678–703.

* "Stahl und Eisen," April, 1893, p. 275.

† In Switzerland the Gothard Railway first adopted mild steel for several superstructures of bridges in the year 1891. In the beginning of 1892, the Northeast Railway followed.

This table shows that basic Martin steel has been preferred for bridge-building, and the same is true in ship-building. The first tests made in the years 1883–85 with Thomas metal by the English for use in ships were not successful, so that Lloyd's in December, 1885, determined not to admit it for such use. Recently, by the request of Mr. Percy G. Gilchrist, the English Admiralty have made additional tests upon this metal, and it was found to weld better, to be less dangerous at blue heat than the acid steel, and to accord with the requirements made in English ship-building. In Germany it has not been largely used for ship-building, because the ship-registering societies seem to consider Martin metal preferable. The writer's opinion, based on comparative tests and the examples of their use, given in the table, is that to-day there is no longer reason to prefer one kind of mild steel to the other, and that the selection should be governed by price alone. If this be admitted it would seem that the Thomas metal would have the preference, as by it large masses of metal can be produced, and the large amount of scrap which is used with the Martin metal must increase its price.

The production in the whole world in 1892 was as follows:

Of Thomas steel.....	2 591 374 tons.
Of basic Martin steel only.....	611 266 "

The distribution of manufacture is given in Table No. 3.

This table shows the important part Germany has taken in the production of this metal. Its production has been almost five times that of England and nearly eight times that of France. There is now no country where the milder metal is not received with confidence as being more desirable than the harder ingot metal for use in structures. Harder metal with a tenacity of 64 000 lbs. (45 kg.) is more easily produced by the acid Bessemer and the Martin process, while the basic open-hearth process is the most satisfactory for producing the lowest and more reliable metals, having a tenacity of 48 000 to 56 000 lbs. (38 to 45 kg.).

The specifications now prescribed by some of the States of Europe for the delivery of mild steel are given in Table No. 4. There are also given some of the earlier requirements, particularly those for vessels and boilers, which are still legal.

This table shows that everywhere in ship-building harder kinds of

metal than in bridge-building are still preferred. These requirements originated from the early English experience when only the acid Martin open-hearth process and the acid Bessemer process were in operation. Moreover, the acid Martin process did not allow the production of softer metals. The later development of the basic process furnishes a cheap and uniform quality of metal with a mean tenacity of 57 000 lbs. (40 kg.).

The introduction of this soft steel in ship-building would be of great advantage to the duration and safety of vessels, and that this is not the case seems to proceed from reluctance to the giving up of old customs and to the widespread error which ascribes to harder metal of high tenacity a greater resistance than to the softer. If this idea were correct it would be possible to lessen the thickness of steel proportionately and to increase the profit in ship-building by diminishing as much as possible the weight of material required. The resisting power of the metal does not, however, depend entirely upon its tenacity, but also upon its toughness, and the combined effort can be reached by the product of the tenacity and elongation, which product is called the resilience of the metal. It will be found that the softer kinds of metal always have a resilience which is very difficult for harder metal and quite impossible for a very hard one.

It is yet uncertain whether steel will in the future be produced of so high a tenacity as 64 000 to 85 000 lbs. (45 to 60 kg.) which shall be as ductile as that now produced having a tenacity of 57 000 to 64 000 lbs. This possibility is not excluded at all events, if the product be entirely free from the impurities of silicon, phosphorus and sulphur, and if the quantity of manganese used be kept low. The resultant product would be a pure mild steel, showing in the best manner the influence of the most important element, "carbon," upon its quality; but, so long as this result has not been reached, ingot metal of low tenacity offers for engineering structures more security than the harder kinds. Such steel stands better than the harder kinds under the operations of drilling, punching and other mechanical operations, and requires less care in manipulation.

On account of its uniformity of texture, ingot steel has been surnamed "homogeneous iron," and it is preferred to wrought iron, for the reason that it has high and uniform elongations in all directions, while the elongation of wrought iron transversely to the direction of

rolling is very small. The yield point of mild steel is one and one-half times to twice that of wrought iron. In a wrought iron bar considerable elongations begin with a load of 23 000 lbs. per square inch, but in a mild steel bar only when the load is at least 34 000 lbs. per square inch. Hence, if we fix the limit at one-half the yield point we may require a work in its use at least one and one-half times that of wrought iron, or, say, 17 000 lbs.

On account of the increase of elongation with the diminution of tenacity of the metal, many engineers have had the opinion that it was sufficient in testing steel to measure only the elongation, and they have insisted upon the highest attainable elongation as being a special requirement. This would exclude material which was too hard, and was, therefore, right in that respect; but there was danger of getting too soft a metal. It was therefore necessary to prescribe both an upper and a lower limit for the tenacity. Table No. 4 shows that most countries desire a steel for structures whose tenacity is between 50 000 and 64 000 lbs. France, however, somewhat unaccountably stipulates at least 60 000 lbs., and an elongation of at least 22 per cent. The requirements made by the Austrians seem somewhat high in consideration of the present state of steel manufacture.

The question of a correct specification for the transverse elongation of test pieces does not seem to be at present quite clear. The former standard by which rolled steel was required to have the same elongation in both directions does not seem to be upheld by experience. The subject has been exhaustively treated for the first time by the committee of the societies of German engineers and German steel-work managers, and is given in their standard specification for the use of mild steel. By request of the committee the great German steel-makers have made tests established upon universal bars and plates.* The averages of these results are given in Table No. 5.

This table shows that steel of 53 000 to 63 000 lbs. (37 to 44 kg.) tenacity and 20% elongation will not give equal stretch transversely and longitudinally without a rejection of about 27% of the universal bars tested, and 12% of the plates. If the requirements be a tenacity of 51 000 to 64 000 lbs. (36 to 45 kg.), and 17% elongation transversely, there will yet be a rejection of 10% in universal bars and 4% in plates.

* The tests were made in the Gutehoffnungshütte at Sterkrade, and the results have been published in "Stahl und Eisen," and in the "Zeitschrift des Vereines Deutscher Ingenieure."

In the writer's opinion the ascertaining of the transverse elongation of mild steel has more of scientific than of practical value, and the requirement of a certain degree of such elongation for various kinds of iron makes the reception of the material more difficult without giving an essentially better material. Aside from careful rolling we should aim to obtain a sufficient ductility longitudinally, and the lower transverse ductility caused by the rolling should be considered as unavoidable. It seems necessary for plates strained in this direction, especially joining plates, to take care that the ductility be as uniform as possible in every direction, and this requirement can be easily reached by suitable transverse rolling. Some authorities have proposed to specify a certain minimum transverse elongation and to neglect testing lengthwise; but this does not seem wise, and it seems sufficient to provide for a testing of the transverse ductility only in plates which will be strained in a transverse direction.

Table No. 4 shows that for rivets steel is demanded of less tenacity and more elongation than that for plates or bars, the tenacity required being 50 000 to 60 000 lbs. (35 to 42 kg.), and the elongation reaching 32 per cent. This requirement evidently arises from the opinion that rivet steel should not be so hard as to make riveting difficult, but there should be no scruple in stipulating the same tenacity and elongation for rivets and screw-plates as are generally prescribed for bars and plates. Still, there are yet some departments which prefer to use wrought-iron rivets in mild-steel structures. This proceeding appears unintelligible.

The fixing of the exact limit of elasticity being difficult, many engineers are content with the ascertainment of the yield point of the metal, which is easily obtained graphically by the use of testing machines. This point is the limit of strain at which the elongation of the bar suddenly becomes observable, even to unaided vision, but is best seen in the diagram taken from the machine. The writer thinks that if the tenacity and elongation are specified, it is not necessary to prescribe the yield point. The resilience or the product of these quantities is characteristic of the quality of the metal as to its properties of ductility and malleability. Engineers who deem it sufficient to determine only the yield point advise that this be made as high as possible, because for use in a structure it would be advisable to use the metal up to the highest safe strain, say one-half the yield point. This is wrong

in so far as it might lead to the raising of the yield point without corresponding increase of tenacity, which would make a brittle material liable to sudden rupture. It is probably on this account that French specifications require the yield point to be not less than one-half or more than two-thirds of the tenacity.

It is questionable whether the chemical composition of steel to be furnished should be specified, and there is a strong opinion among engineers that it is advisable to leave the methods of production in the hands of the steel-makers, while the engineer must watch the process and test the finished metal; there is, however, a difference of opinion on this point. Generally a maximum limit of phosphorus is prescribed, and sometimes of carbon also; the former is more important, and there is no doubt that more than 0.1% of phosphorus makes steel useless for structural purposes.

As to other materials, the physical tests of each charge or blow are sufficient for determining the acceptability of the material, and it seems unnecessary for the inspector to require other chemical analysis.

Table No. 4 shows that the specifications issued by the Swiss Government make the new requirement of a maximum of 0.06% sulphur for steel rivet bars. This is to prevent red-shortness in the rivets, but the same result could be obtained by the customary red-short tests. It is usually advisable for the consumer to encroach as little as possible upon the affairs of the steel-makers. As soon as the production and use of mild steel shall be governed by established rules, then its chemical analysis and composition may with confidence be left with the steel works.

No country is content with testing only for tenacity. All who have had to do with the practical examination and selection of material are convinced that this is not a complete test, and does not necessarily furnish reliable material under all conditions. It would seem, therefore, that other technical tests should be made, such as bending, forging and drop tests, the details of which are known. In Germany, in some cases an effort has been made to introduce a new kind of tenacity test, but without success. This consisted in testing pieces 2 to $2\frac{3}{8}$ ins. broad, with holes $\frac{3}{4}$ in. diameter (usually three), punched cold, at $2\frac{3}{8}$ ins. apart, center to center; the punched pieces were then subjected to the tenacity test, with the requirement that the tenacity must remain within the ordinary limits, and the fracture not show any

change of texture when compared with the fracture of unpunched pieces. The metal would not stand this test, and steel works were perfectly right in refusing to deliver mild steel under this specification. There is no better test or safer one under which to receive mild steel than that of testing blow by blow ; the examination of each blow and of the rolled or finished pieces made from it. The results gained in testing one of the blows cannot be considered as surely indicating the quality of any other. Each blow is likely to show a difference in texture and chemical composition, although usually the difference will almost disappear by good and regular management.

The selection by blows for a small quantity of steel would be impracticable ; but for large structures it should be made obligatory by the specifications.

The inspection of the steel in the state of the ingot should be an internal affair of the steel works, and one with which the engineers have nothing to do.

The last question to be discussed is the treatment and manipulation of mild steel in the shop and the field. It is not long since that it was believed to be necessary to use only hammers of copper in its manipulation. In the year 1888 a commission of French engineers, which attempted to study the question in connection with the use of such material for the bridge over the Danube near Cernavoda, had this opinion. Were this right there would be little prospect of the use of mild steel in the future. Any material requiring such careful handling could not be admitted in general use. There is no apparent reason why material which can be punched without cracking or sensible hardening at the holes, and which stands such severe usage under drop and bending tests, even after subjection to severe cold,* should not be used in an ordinary way.

A good mild steel can be worked as readily as wrought iron in the shop or the field, and even bear still harder treatment. It was, however, often thought necessary to require preliminary annealing to remove the initial strains due to rolling. The annealing is undoubtedly of great advantage to all steel above 64 000 lbs. strength per square inch, but it is questionable whether it is necessary in softer steels. The distortions due to heating cause trouble in subsequent

* Stahl und Eisen, 1892, No. 4 and No. 13. Centralblatt der Bauverwaltung, 1892, pages 68 and 83.

straightening, especially of thin plates; and very long pieces cannot be properly annealed on account of lack of facilities for such purpose at the mills. It cannot be denied, however, that annealing produces greater toughness.

In a general way all unannealed mild steel for a strength of 57 000 to 64 000 lbs. may be worked in the same way as wrought iron. Rough treatment or working at a blue heat must, however, be prohibited. Such treatment cannot be borne by wrought iron, although it does not suffer so much as soft steel. Shearing is to be avoided, except to prepare rough plates, which should afterwards be smoothed by machine tools or files before using (see Table 4, Nos. 7–10). Drifting is also to be avoided, because the edges of holes are thereby strained beyond the yield point. Reaming drilled holes is not necessary, particularly when sharp drills are used and neat work is done. A slight countersinking of the edges of drilled holes is all that is necessary. Working the material while heated should be avoided as far as possible, and the engineer should bear this in mind when designing structures. Upsetting, cranking and bending ought to be avoided, but when necessary the material should be annealed after completion.

The question of riveting is an important one, especially that as to whether the heading of steel rivets is better done by machine or hand work, on account of the danger of working at a blue heat. The riveting of a mild steel rivet should be finished as quickly as possible, before it cools to the dangerous heat. For this reason machine work is the best. There is a special advantage in machine work from the fact that the pressure can be retained upon the rivet until it has cooled sufficiently to prevent elongation and the consequent loosening of the rivet. It is undoubtedly true, also, that rivets beyond 1 in. diameter cannot be well driven by hand work; but such rivets are very seldom used. European bridge-builders, especially those on the Continent, prefer hand work, on account of the expensiveness of the other, and also on account of the difficulty in using a machine upon all the rivets to be driven in the field, which are among the most important in the structure.*

The writer, in closing this paper, desires to state that it is written for the purpose of giving a general view of the use of mild steel throughout the world, to show the importance of further tests, and to

*Notice of the erection of Iron Bridges in America. "Stahl und Eisen," 1891, No. 4.

show that, with the present knowledge that we have concerning the manufacture and use of mild steel, engineers and architects are abundantly justified in accepting it, under proper restrictions, for all classes of structures.

TABLE No. 1.

List of the Most Remarkable Mild-Steel Superstructures of Bridges Built in the Decade 1880–90.

No	Name and site of the Bridge.	Time of erection.	Built or designed by	Dimensions. No. of spans.	Dimensions. Length. Feet.	Specifications for the Material of the Superstructures. Kind of material.	Total weight. Tons. 1 000 kg. = 2 204 lb.	Tenacity in pounds per square inch.	Yield point. Pounds per square inch.	Elongation. Percent.	Remarks.
1.	"Honig" Bridge, Königsberg, Prussia	1880	Frühling, Union Foundry, Königsberg	1	42	Acid Martin	23.5	For bars 71 000 For plates 64 000		20 20	Nos. 1, 2 and 5. Material from the Steel Works of "Phönix" at Eschweiler Aue, Germany.
2.	"Hohe" Bridge, Königsberg, Prussia	1882	Frühling, Union Foundry, Königsberg	1 2	63 88	Acid Martin	130.0	64 000 For rivets 63 500 Shearing strength.		22 22	Nos. 2 and 5, a single lever-draw span and two truss spans.
3.	Railway bridge over the Firth of Forth Queensferry, Scotland	1883–1890	Fowler & Baker Arrol & Co	2 2	1 700 685	Acid Martin	56 000	Compressed members 75 000 to 82 000 Tension members 67 000 to 74 000 Rivets 50 000 to 54 000 Shearing strength.		17 20	
4.	Railway pin-fastened bridge for the Deli-Spoorweg Company, Sumatra	1885	Harkort, Duisburg, Germany	21	33–100	Thomas	156	54 000 to 60 000		20	
5.	"Kottel Bridge, Königsberg, Prussia	1885	Frühling, Union Foundry, Königsberg	1 2	61 65	Acid Martin	108.1	60 000 For rivets 51 000 Shearing strength.		20–22 24	No. 6. Combination of a stiffened suspension bridge, together with an elevated foot bridge.
6.	*Tower Bridge over the Thames, London	1886 Still in erection.	Barry Arrol & Co	1 2	180 270	Acid Martin	13 000	60 000 to 71 000 For rivets 59 000 to 65 000	31 000	20 20	No. 7. The Prague Works furnished since 1887-90, also about 700 tons of small road bridges up to 91 ft. length and 84 tons railway bridges built partly in Thomas and partly in Martin.
7.	Elbe Bridge, Melnik, Bohemia	1887	Bridge Works, Prague	4 1	193 158	Thomas	700	Without specification.			
8.	*Road bridge, Gaska, Japan	1887	Harkort, Duisburg, Germany	2	216	Acid Martin	339	74 000		25	No. 8. Moreover, three road bridges in Thomas for Japan and Brazil. Weight, 142 tons. Specification like No. 4.
9.	Swing bridge over the "Magdeburg" Harbor, Hamburg		Harbor Department	1 2	173 52	Thomas	400	59 000 to 64 000	37 000	25 Contraction 50	No. 9. 0.10 C.; 0.05 P.; 0.35 M.; 0.02 Si.; 0.01 S., were stipulated.
10.	Moldau Viaduct, near Cerwena, Railway Tabor Pisek, Bohemia		Austrian State Railway	3	262	Basic Martin	968	60 000 to 64 000 For rivets 59 000 to 57 000		28–22 32–26	No. 10. Chemical analyses gave as follows: 0.052 – 0.086 C. 0.016 – 0.034 P. 0.162 – 0.210 Mn. 0.016 – 0.050 S. Afterwards followed about 2 600 tons smaller plate and girder bridges in the Austrian State Railway, 262 spans up to 223 ft. length in basic Martin mild steel.
11.	Bridges of Gagnières and Iseron, Paris-Lyon-Med. Railway, France	1887–1889		3	170	Basic Martin		60 000 ± 3 000	Lengthwise 34 000 Transverse 31 000	Lengthwise 25 Transverse 18	No. 11. Elongation measured in 4-in. lengths.
12.	Two foot bridges at the "Jonas" and "Hafenthor," Hamburg	1888	Klönne, Dortmund, Germany	1 1	59 66	Thomas and basic Martin	80	63 000 to 63 000		20	No. 12. The same work furnished in 1888-90 140 tons smaller road bridges up to 82 ft. length Thomas and Martin mild steel combined.
13.	*Bridge of Piazza Fia. over the Tiber, Rome	1888–1889	Savigliano Works, Italy	1	330	Thomas	862	60 000 to 71 000	28 000	18–25	
14.	Thirty-seven railway bridges, Deli-Spoorweg Company, Sumatra	1888–1889	Harkort, Duisburg	37	Up to 200	Thomas	436	54 000 to 60 000		20	
15.	Viaduct do Chã, at São Paulo, Brazil	1889	Harkort, Duisburg	4 1	98 52	Thomas	411	54 000 to 60 000		20	
16.	Bridge on the line from Lons le Saulnier to Champagnole, France	1889		1	230	Basic Martin		60 000 ± 3 000	Lengthwise 34 000 Transverse 31 000	Lengthwise 25 Transverse 18	
17.	"Wallgraben" Bridge, near Marienburg, line Dirschau-Königsberg, Prussia	1889–1890	Harkort, Duisburg	10	59	Basic and acid Martin	260	57 000 to 64 000 For rivets 51 000 to 57 000	35 500	20 25	No. 17 Harkort furnished, moreover, 1890, fifteen railway bridges for the Kinselau Railway in Japan; 13 road bridges for Transvaal, in Africa, and two road bridges near Santos, in Brazil. Total weight, about 1 400 tons Thomas mild steel.
18.	Railway bridge over the Rio Samala, Brazil	1890	Harkort, Duisburg	1	180	Thomas	108	54 000 to 60 000		20	

* Road bridges.

TABLE No. 2.

List of the Most Remarkable Basic Mild-Steel Bridges and Viaducts Built in Europe Since the Year 1890.

No.	Name and site of the bridge.	Time of erection.	Built or designed by	Dimensions.		Specification for the Material of Superstructures.					Remarks.
				No. of spans.	Width, feet.	Kind of material.	Total weight, tons. 1 000 kg. = 2 204 lb.	Tenacity in pounds per square inch.	Yield point, pounds per square inch.	Elongation, per cent.	
1.	Weichsel bridge at Fordon. Line Bromberg-Fordon-Culmsee, Prussia	1891-93	Mehrtens, Gutehoffnungshütte and Harkort, Duisburg.	5 13	328 203	Basic Martin Thomas	11 000	55 500 to 64 000 For rivets 51 000 to 57 000	35 000	20 25	No. 1. P not above 0.10 per cent. Reception blow by blow.
2.	*Swing bridge in the Harbor of Lübeck, road and single track railway	1891	City of Lübeck, Gutehoffnungshütte, Sterkrade	1 1	33 81	Basic Martin	296	54 000 to 63 000			No. 2. The product of the tenacity and the elongation (resilience, Arbeitszifer, capacité de travail) must reach the number 1 000 (for kilogram and square millimeter, or 1 420 000 for pounds and square inches). Chemical composition stipulated.
3.	Donau bridge near Czernavoda. Railway Czernavoda-Küstenche	1891	Saligny, Fives-Lille	1 4	623 460	Basic Martin		60 000 to 67 000 For rivets 54 000 to 63 000	34 000	21-16	
4.	*Elbe bridge Loschwitz-Blasewitz, Saxony	1891-93	Köpcke and Krüger; Königin Marienhütte, Zwickau	1 2	482 203	Basic Martin	3 238	51 000 to 64 000 Rivets wrought iron.	31 000 to 41 000	20-30 Contraction 40-65	No. 4. Highly remarkable stiffened truss suspension bridge, with three hinges, system Köpcke. Nos. 4 and 5. P not above 0.10 per cent ‡
5.	Oder bridge, near Alt-Rudnitz. Line Wriezen-Jädickendorf, Prussia	1891-92	Royal Railway Department, Berlin, Klönne, Dortmund.	3 13	213 115	Thomas	1 300	53 000 to 63 000		20	No. 5. The works of Klönne furnished up to 1893 also several smaller bridges. Weight about 1 000 tons.
6.	Single track Elbe bridge at Niederwartha, Saxony. Line Berlin-Dresden	1891-92	Klette and Bähse, Lauchhammer	3 14	203 69	Basic Martin	1 100	61 000 to 64 000	33 000	20	
7.	*Road bridge over the Sesia at Ghislarengo, Province Novara, Italy	1891-92	Savigliano works	7	138	Basic Martin	640		Like	Nos. 8-9.	Nos. 7-9. In Italy, the Savigliano works furnished also three military bridges, 118 ft. length; weight, 260 tons, in Thomas steel, and numerous road and railway bridges. Among them may be named the viaduct over the Formia (2 spans of 152 ft.), etc. At the present time in Italy, there are finished or in erection about 13 000 tons of mild-steel bridges.
8.	Bridge over the Tanaro. Line Genova-Asti, Italy	1891	Savigliano works	2 1	161 197	Basic Martin	338	60 000 to 71 000	28 000	18-26	
9.	*Road bridge over the Reno, Province Ferrara, Italy	1891	Savigliano works	2 1	121 157	Basic Martin	225	60 000 to 71 000	28 000	18-26	
10.	Three bridges of the lines Schönberg-Hirschberg-Waldheim-Kochlitz A. O. State Railway, Saxony	1891-93	Poppe and Banmann			Basic Martin	1 714	51 000 to 64 000	33 000	20	
11.	*Twenty road-bridges for the Dutch Colonies, India	1891	Colonial Minist., Hague. Gutehoffnungshütte, Sterkrade	20	39 131	Basic Martin	357	Specifications of the Society	of German Steel	Work Managers †	
12.	Viaduct at Lapio. Line Avellino-Rocchetta	1892	Impresa Industriale Italiana Castellamare	3 1	318 285	Basic Martin	1 020		Like Nos. 8-9.		
13.	Four bridges over the Taro. Line Parma-Spezia, Italy	1892	Impresa Industriale Italiana Castellamare	1 3 3	164 118-138 111-141	Basic Martin	1 364		Like Nos. 8-9.		
14.	*Crossing of Waltherstreet over the switching-station, Dresden-Friedrichstadt, Saxony	1892	Prohaska, Lauchhammer	22	33-48	Basic Martin and Thomas	1 438	51 000 to 64 000	33 000	20	No. 14 Only the door bars and the quadrant bars of the columns are of Thomas steel.
15.	Donaubridge at Regensburg. Bavarian State Railways	1892 still in erection.		5	157	Basic Martin and Thomas	650				
16.	*Road bridge over the Caledon, Orange Free State	1892	Harkort, Duisburg	6	148	Thomas	484	54 000 to 60 000		20	
17.	*Moldau bridge at Moldauthein, Bohemia	1892	Bridge-work, Prague	2	196	Thomas	344	Without	specifications.		
18.	Twenty-six bridges of the Nederlands South African Railway Company, Amsterdam	1892-93	Gutehoffnungshütte, Sterkrade	26	16-66	Basic Martin	343	German Standard	Specifications (Table IV).		
19.	Crossing of Bernsel street at Berlin, City Railway	1892	Betriebsamt (Board of Managers) Gutehoffnungshütte Sterkrade	1	62	Basic Martin	118	54 000 to 63 000		20	
20.	Bridge over the Elbe, near Hamburg, line Venlo-Hamburg	1892-93	Royal Railway Department, Altona. Gutehoffnungshütte	3 4	328 75	Basic Martin	2 200	67 000 to 64 000 For rivets 51 000 to 57 000	33 000	20 25-30	No. 20. P not above 0.09 per cent. Reception blow by blow.
21.	Wertach bridge at Augsburg, Bavarian State Railways	1892-93		3	197	Basic Martin and Thomas	380				No. 21. Moreover, since 1891, on the Bavarian State Railways four bridges (of about 600 tons weight), and roof structures for the station and the engine-house (500 tons) at Munich, besides about 1 300 tons smaller bridges have been erected.
22.	Railway viaduct over the Tjitandoel, Java	1893	Colonial Minst. Holland, Harkort, Duisburg	3 2 1	203 26 98	Basic Martin and Thomas	1 045	53 000 to 57 000	Resilience 900 for kilogram per sq. millimeter, or 1 280 000 for pounds and square inches.		No. 22 These well-known works furnished, since 1892, 31 railway bridges for the Roumanian State railways (1 470 tons), 32 railway bridges for the railway Weochang-Hanjyang in China (932 tons), 29 railway bridges for the Kuschin line in Japan (100 tons), and several smaller bridges for Roumania, Sumatra, Brazil, etc. Total about 6 000 tons Thomas and Martin metal from 1892 up to the beginning of 1895.
23.	The crossing over Falken street at Dresden	1893	Gutehoffnungshütte Sterkrade and Lauchhammer	1 1	150 82 71	Basic Martin	307	51 000 to 64 000 For rivets 48 000 to 63 000	33 000	20	No. 23. P not above 0.10 per cent.
24.	Under bridge of the Trankgasse and Johannis street at Cologne	1893	Betriebsamt Köln-Düren Gutehoffnungshütte, Sterkrade	1 1	43 66	Basic Martin	700	63 000 to 63 000		20	
25.	Enz bridge at Besigheim, State Railway, Würtemberg	1893 still in erection.		6 3	118 174	Basic Martin	588				

* Road bridges.
† Vorschriften für Lieferungen von Eisen und Stahl aufgestellt vom Vereine deutscher Eisenhüttenleute. Düsseldorf, 1883.
‡ Bauwerkereche Zeitschrift des Architekten und Ingenieur Vereins 1860, 1861, 1888 und 1889.

TABLE No. 3.

Production of Basic Mild Steel in Various Countries.

Name of the Country.	1891.		1892.	
	Total in tons. 1 000 kg. = 2 204 lbs.	Containing under 0.17 % carbon.	Total in tons. 1 000 kg. = 2 204 lbs.	Containing under 0.17 % carbon.
1. Germany and Luxemburg	1 779 779	1 314 781	2 013 484	1 616 783
2. England	436 261	350 818	406 839	317 585
3. France	255 401	173 880	287 528	196 190
4. Austria-Hungary	221 212	95 907	288 122	212 408
5. Belgium, Russia and United States	187 882	111 172	206 667	129 028
Total	2 880 535	2 046 558	3 202 640	2 471 992

TABLE No. 4.

Essential Specifications for Mild-Steel Structures in Several Countries.

No.	Name of the authority or society.	Kind of material.	Tenacity Tests. Tenacity. Pounds per square inch.	Tenacity Tests. Elongation in 8 ins. Per cent.	Tenacity Tests. Yield point, pounds per square inch.	Other Tests. For bars and plates.	Other Tests. For rivets and screws.	Remarks.
		A. Specifications for Ships and Boilers.						
1....	English Admiralty.....	Plates for vessels and boilers..	57 000 to 66 000	20				
2....	German Admiralty.....	Plates for boiler shells.....	60 000 to 67 000	20				
		Fire-plates for boilers.....	60 000 to 64 000	20				
3....	Lloyd's Register, England..	Plates of vessels.....	63 000	20–25				
		Plates for boilers.....	58 000 to 67 000	20				
		Plates of vessels.....	63 000 to 71 000	16				
4....	Bureau Veritas, France. ...	Plates of vessels.....	60 000 to 71 000	20	.....	No. 4. For plates of $\frac{1}{4}$ in. (6.5 mm.) thickness, or less 16% elongation sufficient. Recommend fire-plates not above 62 600 lbs. tenacity and not less than 22½ elongation; for undulated fire-plates, the like not above 57 000 lbs. and 25 per cent.		
		" for boilers.....	55 500 to 68 000	26–20				
			48 000 to 54 000	32–27				
5....	Board of Trade, England....	Plates for boiler shells.....	60 000 to 71 000	20				
		Fire-plates for boilers	48 000 to 67 000	20				
6....	Germanic Lloyd, Germany.	Plates of vessels.....	58 000 to 70 000	20	.....	No. 6. Recommend fire-plates not above 60 000 lbs. and not less than 22,5 per cent. For plates above 1 in. (25 mm.) thickness, the admissible greatest tenacity shall be taken for each increase of $\frac{1}{16}$ in. (2 mm.), about 710 lbs., less than the numbers of the table.		
		" for boilers.....	50 000 to 68 000	26–20				
		B. Specifications for Bridges and other Buildings.						
7....	French Ministry of Public Works, August, 1891.....	Plates and bars at least.....	60 000	22	30 000 to 40 000	No. 7. Bending tests with pieces which have been punched and quenched, in such manner, that the ends of the testing-piece touch together without crack.	.....	No. 7. All rivet holes are to be drilled or reamed about $\frac{1}{16}$ in. (1 mm.) after drilling. All shearing faces (Schnittflächen) are to be trimmed at least $\frac{1}{32}$ in. (1 mm.).
		Rivets " "	54 000	28				
8....	Austrian Ministry of Commerce, 1892	Plates and bars.....	60 000 to 64 000	28–22	.....	No. 8. Customary bending test and quenching test with test pieces of 2 — 3½ in. (50–80 mm.) breadth and 180° angle of bending : *a.* For material of 60 000 lbs. tenacity, the sides of the test-pieces must quite touch one another after bending. *b.* Material of 64 000 lbs. tenacity, must bend round a pin, whose diameter is equal the thickness of the piece. The same test to be made round a pin whose diameter is five times the thickness of piece, with nicks made in the piece by a stroke of chisel up to $\frac{1}{10}$ thickness of piece transverse to the rolling direction. B. *a.* Angle of bending 150°. *b.* Angle of bending 90°, causing no suddenly entering thorough crack to appear. Plate pieces heated red must be bent quite together over a sharp edge.	No. 8. Ordinary bending test in such a manner that the sides of piece touch quite together. Bending round a pin whose diameter is like that of the rivet, and re-straightening up to an angle of 90°. Upsetting test at red heat up to one-third rivet length. Upsetting test at red heat and forging at blue heat.	No. 8. Only basic Martin mild steel admitted. Shearing faces to be trimmed about $\frac{1}{16}$ in. (2 mm.). All rivet-holes to be drilled.
		Transverse.....	50 000 to 64 000	26–20				
		Rivets	50 000 to 57 000	32–26				
		Ingot steel for bearings.....	81 000	10 at least.				
9....	Standard specifications of German societies, 1892...	Bars and plates (of $\frac{1}{2}$ to 1½ ins thickness) lengthwise.....	53 000 to 63 000	20	.....	No. 9. Quenching test, the diameter of the loop at the bend in test of pieces cut lengthwise must be no greater than the thickness of the piece, and if cut transversely, not more than twice that thickness. Red short test. The test-piece must be heated to redness, and forged to $\frac{1}{4}$ x 1.6 ins. (6 x 40 mm.), then punched by a punch $\frac{1}{16}$ in. (20 mm.) diameter. The hole to be enlarged afterward to 1,2 ins. without cracking.	No. 9. Quenching test. Diameter of the loop (Schleife) to be half of the thickness of rivet. Upsetting test up to one-third of the rivet length.	No. 9. Reception blow by blow, if stipulated. From each blow there are to be tested three pieces at most and from each 20 pieces a single piece. Otherwise there are to be tested from each 100 pieces five, and at most from each 2 tons of the same bars a single piece.
		Transverse.....	51 000 to 64 000	17				
		Rivets and screws.....	51 000 to 60 000	22				
		Ingot steel for bearings.....	64 000 to 85 000	10				
10....	Federal Council of Switzerland	Bars and plates, lengthwise...	57 000 to 64 000	Resilience* 1 280 000	.....	No. 10. Ordinary bending and quenching tests. Pieces 2 ins.(50 mm.)breadth. Diameter of pin equal two-thirds thickness of piece in longitudinal sections, three-halves thickness for transverse sections, and double thickness for universal bars. Red-heat bending-test. Sides must touch quite together. Quantity of P not above 0.1 per cent.	No. 10. Bending test as for bars. Upsetting test. Length of rivet to be twice the diameter and to upset up to one-third. Quantity of sulphur not above 0.06 per cent.	No. 10. Obligatory reception blow by blow. From each blow are to be ascertained the quantities of M. and P., for rivet bars also of S. From each 10 blows also C, Si and S.
		Universal and flange plates, Transverse.	51 000 to 64 000	1 140 000				
		Rivets and screws.....	61 000 to 60 000	1 420 000				

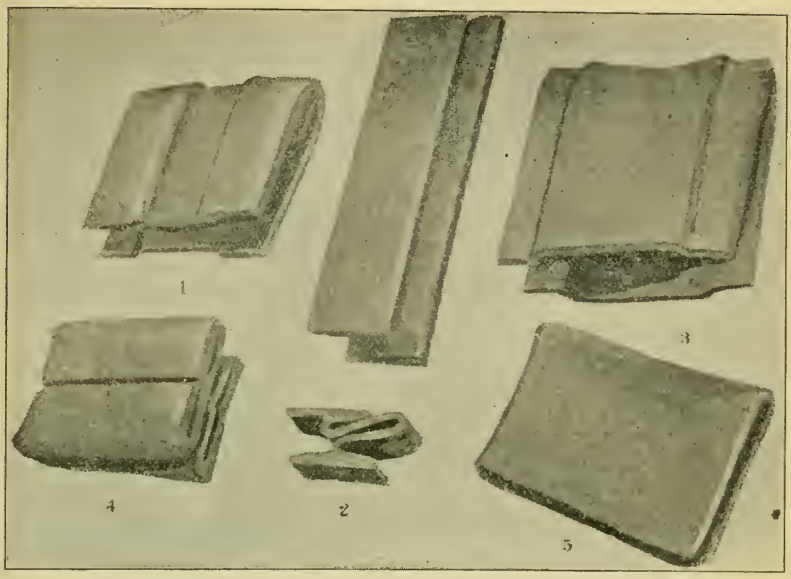
* Resilience (Arbeitsziffer, capacité de travail) or product of tenacity by elongation, expressed in pounds per square inch.

TABLE No. 5.

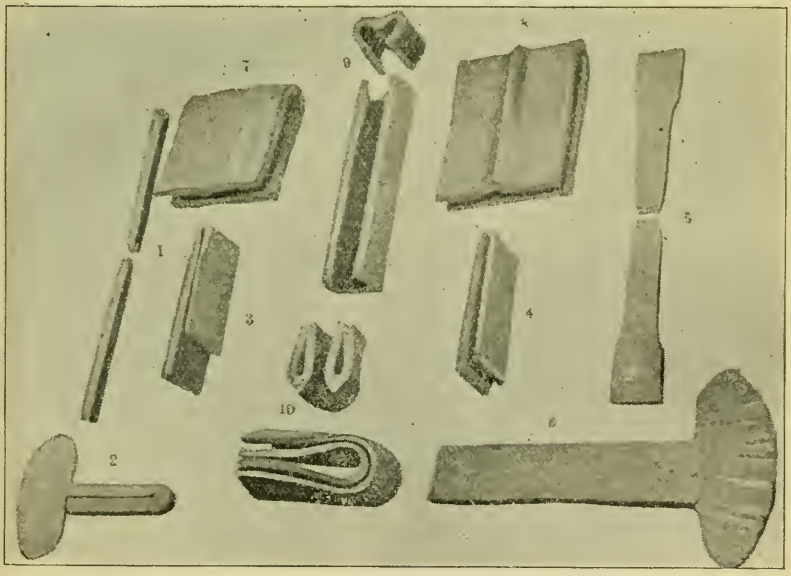
RESULTS OF TRANSVERSE TESTS, WITH PLATES AND BARS, GIVING NUMBER OF TEST PIECES REJECTED FOR FAILURE OF TRANSVERSE TESTS.

Quality of material tested.	From universal plates. 234 test pieces in length. 234 test pieces transverse.	From plates. 193 test pieces in length. 193 test pieces transverse.	From bars. 40 test pieces in length. 40 test pieces transverse.
Pounds per square inch.			
Tenacity 53 000 to 63 000, and 20% elongation.....	63 pieces = 27.00%	22 pieces = 11.42%	1 piece = 2.5%
Tenacity 53 000 to 63 000, and 17% "	43 pieces = 18.37%	17 pieces = 8.81%	1 piece = 2.5%
Tenacity 51 000 to 64 000, and 17% "	24 pieces = 10.00%	7 pieces = 3.78%	1 piece = 2.5%
Tenacity 51 000 to 64 000, and 15% "	19 pieces = 8.12%	3 pieces = 1.60%	0 piece = 0.00%

PLATE I.—BASIC MARTIN O.-H. STEEL FROM THE FORDON BRIDGE.

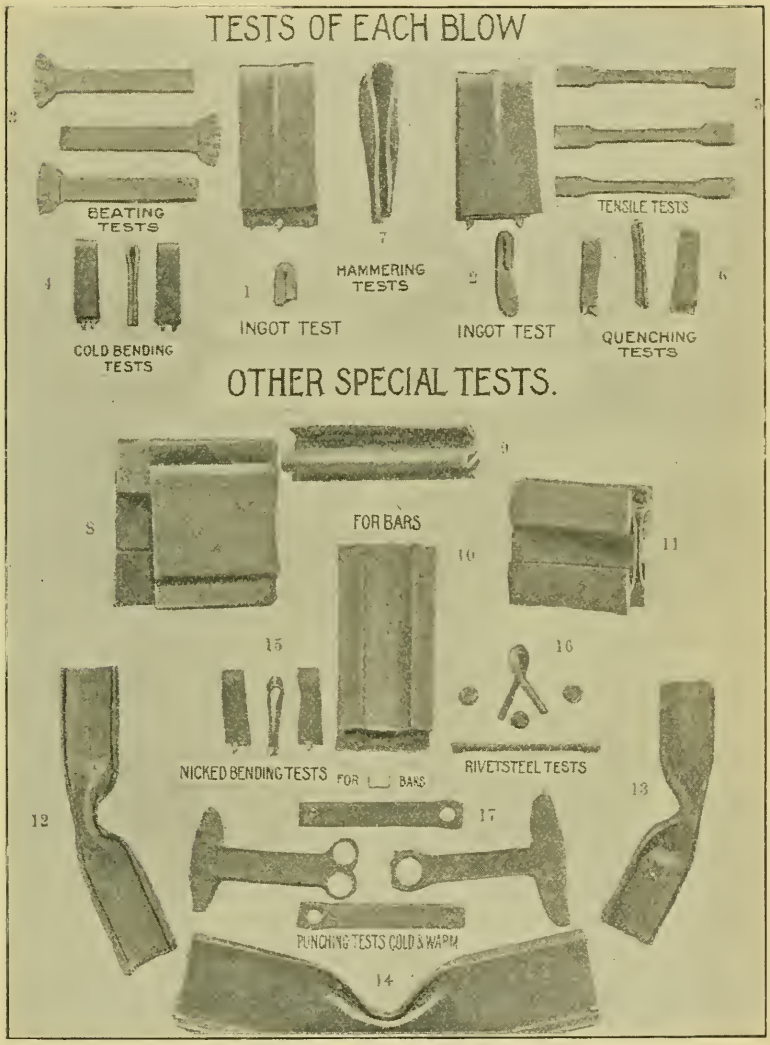


1-5. Special Drop and Bending Tests.



1-2. Ingot Tests. 3-4. Bending Tests. 5. Tensile Test. 6. Beating Test. 7-10. Special Drop and Bending Tests.

PLATE II.—THOMAS MILD STEEL FROM THE FORDON BRIDGE.



AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

643.

(Vol. XXX.—October, 1893.)

THE USE OF BASIC MILD STEEL AS MATERIAL FOR CONSTRUCTION IN GERMANY.

By C. WEYRICH, Engineer of River and Harbor Board at Hamburg, Germany.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

Among the iron-ore-producing countries of the world, Germany holds the third position. The United States and Great Britain, being respectively first and second, are followed closely by Germany, while Spain follows at a much greater distance.

The total produce of iron ore for all countries in the year 1890 amounted to about 67 000 000 tons, distributed as follows :

	Tons.
United States	17 500 000
Great Britain	14 000 000
Germany	11 500 000
Spain	5 000 000
All the other countries	19 000 000
Thus making a total of	67 000 000

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

About 40%, or 27 000 000 tons, of pig iron was made from this quantity of iron ore, and was converted into :

	Tons.
Wrought iron	7 500 000
Steel	12 000 000
Castings	7 500 000
Making a total of	27 000 000

Of these 12 000 000 tons of steel, 2 500 000 tons were basic or Thomas steel, of which 2 000 000 tons contained less than 0.17% carbon.

The total quantity of 2 500 000 tons of basic steel was manufactured by the different countries as follows :

	Tons.
Germany	1 500 000
Great Britain	500 000
France	250 000
Austria	125 000
Belgium, Russia and United States	125 000
Together	2 500 000

The commercial value of the iron ores produced in Germany in the year 1890 was estimated to be about 48 000 000 marks, or about 4.2 marks per ton. The export of Germany in the year 1890 amounted to nearly 2 000 000 tons, from which amount France and Belgium received about equal parts. The import reached nearly 500 000 tons, chiefly of Spanish origin.

The total output of pig iron in Germany was 4 658 000 tons, and of these there were used in the manufacture of—

	Tons.
Steel	2 256 000
Wrought iron	1 581 000
Cast iron of 2d fusion	781 000
The rest being 1st fusion, old iron and graint of pig iron from the blast furnace cinder	40 000
	4 658 000

The manufactured products were :

	Tons.
Steel	1 614 000
Wrought iron	1 486 000

Or in detail :

	Steel. Tons.	Wrought Iron. Tons.
Rails	560 000	11 000
Ties	130 000	16 000
Tires, railway axles, wheels	93 000	16 000
Merchant iron, plates, etc	515 000	1 258 000
Wire	217 000	122 000
Various materials	99 000	63 000
	1 614 000	1 486 000

Since this time the change in the production in Germany has not been very marked, the quantities being for—

	Pig Iron. Tons.
1890	4 658 000
1891	4 452 000
1892	4 793 000

Other countries appear to have fluctuated in like manner; for instance, the United States, the most productive country, shows also a decrease for the year 1891, which was balanced in 1892.

The output was for the three consecutive years :

	Pig Iron. Tons.
1890	9 349 000
1891	8 412 000
1892	9 303 000

It may therefore be said the output for 1890 may be taken as a fair average for the present production. Only in the production of Thomas steel has a considerable progress been made at the expense of wrought iron.

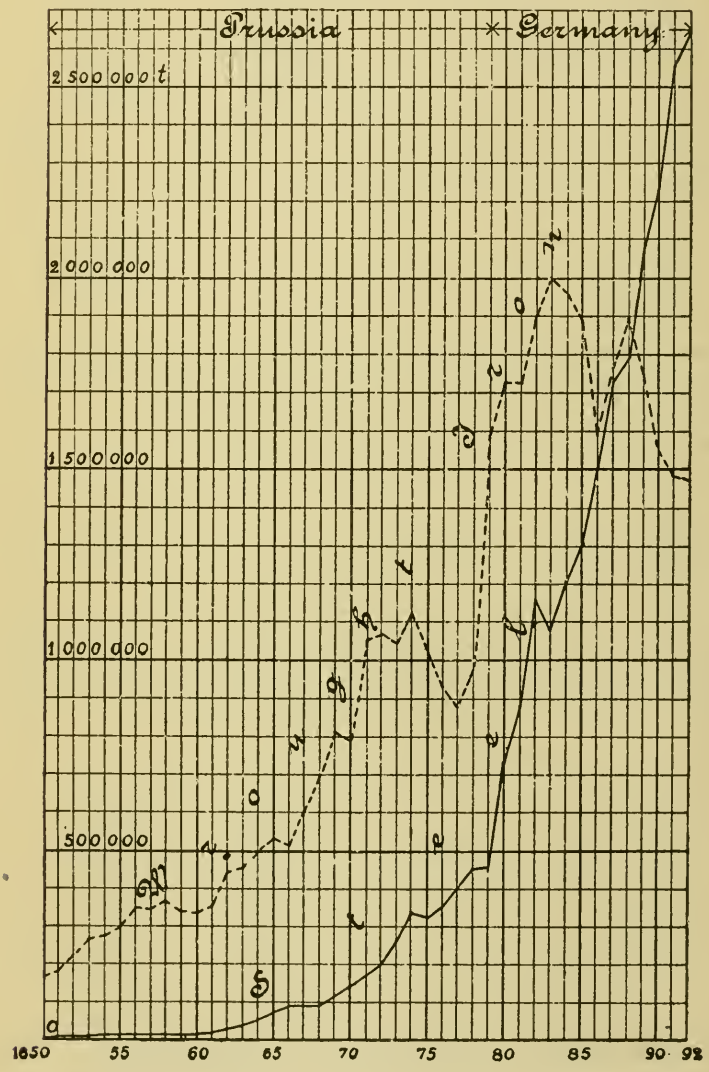
The returns for 1892 in proportion to 1890 are as follows :

	1890. Tons	1892. Tons.
Germany	1 500 000	2 000 000
Great Britain	500 000	500 000
France	250 000	250 000
Austria	125 000	250 000
Belgium, United States, Russia.	125 000	250 000
Thus making a total of	2 500 000	3 250 000

It will be seen that the greatest increase was in Germany.

The development in the production of wrought iron and steel in Germany since 1850 is shown in the diagram.

Production
of wrought iron and steel
from 1850 until 1892.



Bessemer's invention of 1861 and the introduction of the acid Martin process in 1865 gave only a moderate impulse to the somewhat unimportant German steel industry. The German ores could not be worked in the acid process, as they contained too much phosphorus, and had, therefore, to be mixed with imported ores, principally of Spanish origin, in order to produce an available metal. In this way the existing demand for steel rails, for instance, was covered. But the price of the acid steel being too high, it could not compete successfully for purposes where wrought iron was usually applied. It was through the celebrated invention of Thomas and Gilchrist, in 1878–79, which enabled the phosphorus to be extracted by means of fitting a basic lining to the converter, that a great change in the production took place.

Since that time the German ores have been easily converted into steel, the consequence being a great increase in the production of steel. In the year 1880, the year following the introduction of the Thomas process, the produce of steel was double that of the previous year, viz., 750 000 tons against 470 000 tons; and in the year 1892 the output reached 2 650 000 tons, or six times as much as that of the year 1879.

During the same period a great fluctuation took place in the manufacture of wrought iron. Until the year 1883 the production of wrought iron was still increasing, and reached its highest level with 2 002 000 tons. After that time the production, after a short period of revival, gradually diminished to 1 470 000 tons in the year 1892.

A visit to the German steel and iron works confirms this development. While the steel works are increasing in number and extent, the puddling hearths disappear more and more, so that it appears to be a matter of time only when the production of wrought iron will be of secondary moment.

In 1886 the basic process was supplemented by the introduction of the basic Siemens-Martin process. By this new method new friends were gained for the use of steel. The stormy flow of the Thomas process was found objectionable by many, and there were no means of influencing its course. The Siemens-Martin process requires more time to carry it through; it allows a regulation in the formation of the flow. At present both processes are used promiscuously. The majority of the consumers does not attach importance to the method

in which the material is manufactured, while some prefer Siemens-Martin steel.

The production of Bessemer material has scarcely kept on the same level as in earlier times. At present about 12% of the whole quantity of steel is made by the acid process. About 13% of the remainder is Siemens-Martin steel, and 87% Thomas steel.

The respective figures for 1892 are :

	Tons.
Bessemer steel	310 000
Basic Siemens-Martin steel	300 000
Thomas steel	2 040 000
Thus making a total of	2 650 000

During the first years following the introduction of the Thomas process, it was the hard kind of material only that was produced, and it was principally used for rails. The production of rails in the year 1882 was 400 000 tons, of which, of course, the greater part was made by the Bessemer process. But the advantage of the Thomas steel is its softness, it is more easily manufactured, does not suffer by riveting, or accidents, does not injure in working, and is not in any way endangered by exposure to frost, at least such as is likely to be experienced in Germany, the only change in its physical properties being perhaps a very small loss of tenacity. With the progress in the manufacture of Thomas steel, its special properties enabled it to gain new ground; and this was promoted by successful improvements in the new process and the plant employed, which were the means through which a good mild material could be produced with perfect certainty. In the earlier stage Thomas steel was used principally for engineering and ship-building purposes. The German navy particularly became a great consumer. In this application the material is required to have an ultimate tensile strength of between 45 to 50 kg., or 64 000 to 71 100 lbs. per square inch; whereas, for rails, a tensile strength of 50 to 60 kg., or 71 100 to 85 300 lbs. per square inch, is demanded.

With further progress in manufacturing milder kinds of steel, this material drove away wrought iron from the field of iron constructions and soon took possession of the iron market for all general purposes, entirely excluding the competition of Bessemer steel, as this material could not be manufactured of such mildness as is required for these purposes.

The following table shows the production of the two kinds of rolled iron.

	Wrought Iron. Tons.	Mild Steel. Tons.
1882	about 1 000 000	59 000
1883	1 020 000	about 75 000
1884	1 044 000	97 000
1885	1 071 000	about 120 000
1886	1 076 000	139 000
1887	1 264 000	234 000
1888	1 276 000	349 000
1889	1 357 000	496 000
1890	1 258 000	515 000
1891	1 179 000	603 000

It will be seen that during the 10 years from 1882 to 1891 the production of rolled iron manufactured from mild steel increased tenfold, viz., from 59 000 to 603 000 tons, while the manufacture of the same article from wrought iron—at present the only use for this material—advanced very slowly, until in the year 1889 a point was reached from which there was a constant decrease. But as material suitable for the construction of bridges, the use of mild steel would not advance, although its use for that purpose was urgently desired, not only on account of the great demand, but also as the admission of this material, as such, would at once destroy all scruples against its use.

The slow advance corresponds with the characteristic conservatism of Germany. While in America, the country of the most unhindered inducement, novelties are always welcomed, a failure is soon excused and furnishes experience only, in Germany such mistakes are not at all admissible. He that would encounter such a failure, would suffer damage to his reputation. This accounts in a great measure for the extreme backwardness in the adoption of new material. Another drawback is that all public constructions require the approval of the government authorities, who are bound by existing instructions and do not possess the power to introduce novelties. There were still more difficulties to be encountered. The opinions as to the material to be used in the construction of bridges differed. As bridges were not frequently built, experience could not readily be gained.

Further there exists very little intercourse between the civil engi-

neers responsible for the construction and the steel manufacturers. The civil engineer in charge has no time to study the practice of steel manufacturing; he, therefore, lacks the necessary knowledge, being satisfied when he receives the material, which is offered to him as tested according to the specifications; he, therefore, does not know whether all of his wishes are at all times practicable.

The steel-maker lacks a general view of what is required and in what direction the tendency for changes in the requirements are moving, and therefore is not able to change his management accordingly. These circumstances in Germany, contrary to other countries, opposed the progress of mild steel for the construction of bridges, so that it gained ground very slowly. For these reasons it was only very recently that an agreement was arrived at, that the soft material only was to be used. This material is the only one the steel-makers can produce with the greatest possible safety and give to the civil engineer the desired guarantee.

The first very modest trials to build bridges of mild steel were made in Königsberg, in the years 1882 to 1884, by the building of two road bridges of moderate size. Then followed the erection of a bridge of larger dimensions in Hamburg, which was executed by the author of this paper for the State of Hamburg. The bridge served for the use of pedestrians and carriages, and also for two sets of railway lines, and was the first railway bridge in Germany made of mild steel. After the successful completion of this work, more bridges were built of mild steel in Hamburg.

The Prussian Railroad Board then used the same material. At first it was tried with a small railway bridge, and a bridge of larger dimensions is at present under construction, crossing the River Weichsel. Recently, other railway boards have also begun the use of this material in bridges, so that the use of mild steel for these purposes has found the right direction.

All these expectations will be advanced, nevertheless, by creating uniform specifications for mild steel constructions.

The three great associations of German engineers, viz.:

(1) "Der Verband deutscher Architecten und Ingenieur Vereine,"

(2) "Der Verein deutscher Ingenieure,"

(3) "Der Verein deutscher Eisenhüttenleute,"

constituted in the year 1891, a committee for examining this question,

and this committee formulated a standard specification, which was agreed to by the above-mentioned associations, and as the assent has been promised by the German official authorities, this will become the standard for Germany.

Before they entered into the deliberation, the question was raised whether these specifications should extend to the material used for ship-building. But everybody was convinced that this was not advisable. The material used in Germany and abroad for ship-building purposes is of harder quality than the mild steel used for buildings, and it would be a serious matter to bring forward an alteration for Germany only, on account of the classification of ships.

The principal conditions agreed upon were as follows :

The mild steel must show a smooth surface without flaws and blisters and shall not have cracks on the edges and must be sound in all parts.

If an examination by charges is agreed to, every sample put before the official has to bear the same mark as the charge.

Of each charge submitted for approval only three samples are allowed to be taken; the highest number allowed out of each 20 pieces, or 20 pieces on which work has been commenced, is one sample; these are to selected and to be used for testing under the following conditions:

If an examination by charges was not agreed to, the selection for testing of five samples out of each 100 may be allowed; the highest number allowed is one sample out of each 2 000, or commenced 2 000, kg. of the material laid down for approval. In both cases they should use, if possible, the waste ends of the material. If all the tests correspond with the specified conditions, the respective material must be recognized as accepted. For each unsatisfactory sample of the respective material two others may be taken. If any sample of these should fail to satisfy the conditions laid down, the whole quantity of material must be rejected.

The following specifications are established for material, which is from 7 to 28 mm., or $\frac{1}{4}$ to $1\frac{1}{8}$ ins., thick; for other thicknesses special agreements are to be made.

A. Machine Tests.—The material is to have an ultimate tensile strength in the longitudinal direction of not less than 37 kg. per square millimeter, or 52 600 lbs. per square inch, and not more than 44 kg. per square millimeter, or 62 600 lbs. per square inch, with an elongation of not less than 20 per cent.

In the transverse direction the tensile strength to be from 36 to 45 kg. per square millimeter, or 51 200 to 64 000 lbs. per square inch, the elongation being at least 17 per cent.

For rivets and bolts a tensile strength of not less than 36 kg., and not more than 42 kg., per square millimeter, or between 51 200 and 59 700 lbs. per square inch, the elongation being 22 per cent.

B. Various Tests.—1. Flat-iron, special iron and plates.

a. Bending tests.

Samples are to be taken both from the longitudinal and transverse direction and to be heated properly to a low cherry-red and to be quenched in water of about 28° Celsius, and then to be bent to a loop, with an inner diameter equal to the thickness of the sample for longitudinal tests and double the thickness of the sample for cross tests. The longitudinal tests shall not show any cracks; whereas, the cross tests may show such on the surface of no great importance.

b. Test for red-shortness.

A sample of 6 mm., or ¼ in., thickness, and about 40 mm., or 1½ ins. breadth is to be forged while red hot and to be punched in this condition with a tapered punch 80 mm., or 3 ins., long and 30 mm., or 1¼ ins., in diameter at the large, and 20 mm., or ¾ in., in diameter at the small, end. The 20-mm., or ¾-in., hole is to widen to 30 mm., or 1¼ ins., in diameter, without causing any cracks.

2. Rivets and bolts.

a. Bending tests.

Round bars, after having been properly heated to a low cherry-red and quenched in water of about 28° Celsius, must be capable of being bent to a loop of which the diameter is not greater than half the diameter of the sample, and must then not show any cracks.

b. Riveting tests.

A sample of a screw bolt or rivet with a length of double its diameter is to be heated uniformly to the corresponding heat at which it will be used; it then is to be upset to one-third of its original length, without showing any cracks.

The samples of bars to be used for tensile tests are to be cut, when cold, from the piece submitted for approval and to be worked in a like manner.

The damages which may result from shearing or from punching the material are to be removed. Annealing is to be avoided, if the piece itself is not to be annealed during working. The skin of the samples taken for testing is not to be destroyed.

As a rule the samples ought to have a length of 200 mm., or 8 ins., and 300 to 500 sq. mm. or ½ to ¾ sq. ins., of transverse section.

Round bars of less than 20 mm., or ¾ in., diameter should have a length equal to 10 times the diameter.

At each end of the proper testing length the samples are to be kept longer about 10 mm. or ⅜ in., with the same section.

When the test has been made, and the fracture taken place above or below the middle third part, and if it then does not show the expected elongation, the test must be repeated.

The machine for testing must be open for easy inspection for exactness.

The samples to be selected for bending tests are to have 30 to 50 mm. or 1¼ ins. to 2 ins. breadth, or a round bar of such a thickness as will be used afterwards.

All samples are to be prepared when cold, and the edges are to be smoothed off.

Every working of the mild steel is to be done cold, or at least in red-hot condition. All work under conditions between these two temperatures is to be avoided as much as possible. If this cannot be avoided, then the finished material is to be annealed in a proper way.

If mild steel is sheared, then the material next to the section is to be taken off by planing or shaping to a depth of 2 mm. or $\frac{1}{16}$ in., excepting in more unimportant parts, such as lining pieces, etc.

All bolt and rivet holes are to be drilled, with the exception of those in lining pieces, which may be punched. The burr in the holes is to be carefully removed before fitting and riveting the pieces.

Riveting at the place of erection is to be avoided when possible.

Rules regarding the manner in which the manufacture of mild steel is to be performed have not been proposed.

Mild steel of either basic or acid origin may always be passed for application. The Bessemer steel of course will not be recognized as a suitable material, as it cannot be produced under the above-mentioned conditions.

The requirements of the specifications are that the mild steel shall shall be worked in the same state as when it left the rolling mill. A material of the proposed mildness does not require annealing.

Through annealing the material is liable to warp, and therefore has to be straightened again, the consequence being, if this is not done with good tools, that new strains will arise instead of all strains being removed.

The acceptance of mild steel by charges was left to be a matter of judgment, and accordingly the application of marks on the rolled steel will not be insisted upon. A great deal of the ordinary steel which consumers require to cover their current need, will be taken from the ironmonger and not bought from the steel works; for this reason the acceptance by charges will not be practicable. This material cannot be measured with the scale of conditions of the standard rules, as this would only tend to make the material unnecessarily costly. Material manufactured with the standard rules, being used for all more important purposes, would be warranted only by putting on the

number of the charge, and at the same time adding on it the name of the manufacturing steel works in order to base any claims on its identity. To these two marks a third one should be added, as a sign that the material will conform to the standard rules laid down. Putting all three stamps on the material, many finishing rollers being used both for wrought iron and steel, would be impracticable for different reasons. Relief stamps may hinder proper fitting when connections are made with straps, and stamps sunk in will cause a reduction in the area of the transverse section. It will be better, therefore, to do without any marking if possible. The standard rules are laid down in order to insure a reliable material for buildings requiring an absolute certainty. In such cases the material will not be bought from the ironmonger, but is to be manufactured under special inspection according to the standard rules, and is then to be tested by charges, as previously mentioned. The expense of this method of testing seems to be justified for such purposes, and is comparatively of little moment, as in most cases large quantities are dealt with.

The testing conditions relate only to thicknesses of from 7 to 28 mm. or $\frac{1}{4}$ in. to $1\frac{1}{8}$ in., as for all purposes outside of this limit the conditions would scarcely be possible. Material below 7 mm. or $\frac{1}{4}$ in. will be cooled down too much during rolling, and will consequently be too hard, the strength increases, while the elongation diminishes. A material above 28 mm. or $1\frac{1}{8}$ in. thick has not all been properly worked, and will therefore not answer the conditions laid down.

The most important factor is the strength in combination with the elongation of the material. It has been stated already that only the milder kind of steel can be used. Only a material that is free from hard places can give a guarantee as to its security. The hardening destroys the tenacity and therefore makes the material unreliable in use, and working at different degrees of temperature will create internal strains. Steel of an ultimate tensile strength of 50 kg. or 71 100 lbs., supposing normal chemical composition, approaches the limit of hardening. The highest limit for the ultimate tensile strength was made 62 600 lbs., so as to keep clear of all inclination to hardening. But the lowest strength has also its limit, because the material will become too soft and difficult to be worked. The lower limit of 23 tons seems practicable, and this leaves a play of 10 000 lbs. in the production of mild steel for building purposes. Perhaps this limit may be deemed too

wide by many, as materials of 52 600 and 62 600 lbs. ultimate strength are relatively of different nature. But it must be said that with equal chemical composition, and independent of the manufacturing works, the material appears to still have great fluctuations in the limit of strength.

The heat of the charges at the beginning and the end, the position in the hearth or ingot, the subsequent working and treatment, the ratio of the surface of the finished material to the transverse section, the pressure during rolling in the different parts of the material, as in the flanges or the web of special steels, the length of the manufactured pieces, as well as the place from which the sample is taken, either from the middle or ends, all may influence the strength of the material; for instance, there exists a certain difference of strength between plates and special steel in consequence of their being rolled in different ways, although the chemical constitution is the same in both cases. Taking all these circumstances into consideration it may be deemed necessary to allow the stipulated limit.

For bars to be used in tension and compression, no different material is prescribed, although it is not to be mistaken that safety in compression makes it desirable to introduce a harder material. The elongation was fixed at the limit of 20% in order to insure a material of sufficient tenacity. A material of this quality can be made as easily by the Siemens-Martin process as by the Thomas process with the ordinary rolling plant.

Special specifications were stated for tests in the transverse direction of the material. The supposition that mild steel is to such an extent homogeneous that there are no differences as to its properties in the longitudinal or transverse direction cannot be definitely accepted. The unavoidable air holes which are present in ingots before rolling takes place, will be lengthened longitudinally as long as rolling is done in the same direction; whereas, they will be crushed by being rolled transversely and longitudinally. In the first case the cross tests taken from this material would not give such satisfaction as the longitudinal tests of the same material, the elongation being at the same time diminished. For this reason, and in order to avoid complication, there will not be required the same elongation in cross-tests, either for plates or for special steel shapes.

The results of tensile strength developed in cross tests vary so much

that it has been considered necessary to give a wider margin in every direction, of about 1 400 lbs., from 51 200 to 64 000 lbs. per square inch.

For rivet and bolt steel a material of greater mildness than the construction material is recommended. The enormous changes of form developed during working, especially in rivets, make it necessary to have a material of the highest ductility and tenacity. For this reason the elongation and the corresponding strength have been fixed as laid down in the specification.

Besides these conditions regarding strength and elongation, normal rules are laid down relative to mechanical tests as to working, in order to ascertain in another way whether the material is too hard, or too mild.

The total testing length of the samples is fixed throughout at 200 mm., or 8 ins. Only with round steel of less than 20 mm. or $\frac{3}{4}$ in., diameter, should the length be reduced to 10 times the diameter. If the length of 200 mm. or 8 ins., should be insisted on, the prescribed elongation could not be maintained throughout, because, with a decrease in the sectional area, the elongation at the breaking point also diminishes.

As a rule the sectional area should be between 300 sq. mm. (equal $\frac{1}{2}$ sq. in.), and 500 sq. mm. (equal $\frac{3}{4}$ sq. in.), larger differences being excluded. The elongation would be influenced too much. In these limits there already exist large differences. A bar of 300 sq. mm. or $\frac{1}{2}$ sq. in., sectional area, which has 20% elongation, would otherwise under equal conditions have 23% elongation, having a sectional area of 500 sq. mm. or $\frac{3}{4}$ sq. in. Therefore it is not advisable to have too great a margin.

If the fracture takes place outside the middle third of the tested length, the samples are to be rejected in case the elongation is insufficient, because under these conditions the elongation cannot be taken into full account.

If possible the skin of the material should not be broken, as it has been found by experience that this diminishes the ratio of elongation.

There are no conditions laid down relative to the sectional form of samples, although the amount of elongation is influenced by the form of section. Yet such form must be taken into consideration, and the conditions laid down for elongation are so arranged as to enable the same results to be obtained with less favorable sectional forms.

With regard to the chemical composition of mild steel, no rules have been laid down. It will be accepted that, if the physical tests as fixed by the specifications are satisfied, then, also, will the chemical composition be proper. But this does not apply to every case. For one reason the chemical admixtures cannot be gauged by mechanical tests, and for another reason it may be possible to conceal bad qualities by a change in the mixing of different elements to that extent that they will not be discovered when tried under the prescribed tests, while in other respects they may appear afterwards. At all events, specifications concerning chemical composition would considerably complicate the conditions as being difficult to execute. It may be said, that the ores of Westphalia and the Rhine district, worked to basic mild steel, furnish a finished material which has the following chemical compositition :

Carbon	0.09%
Manganese	0.35%
Phosphorus	0.05%
Silicon	0.01%
Sulphur	0.04%

The sulphur is rather high, as the German ores contain a good deal of it, and as the basic process cannot remove it sufficiently. But recently, however, with the introduction of mixing apparatus in which all the runnings of the blast furnaces will be gathered, a process has been introduced through which a considerable amount of sulphur is being expelled, so that at the present time the basic mild steel can be produced of greater purity.

With regard to the allowed strain in mild steel no specifications could be agreed upon, as this would exceed the purpose and should be left entirely to the constructor.

With all bridge constructions hitherto made in Germany, the conditions for wrought iron have been employed throughout, which allow in general a strain to be taken as high as 7 kg. per square millimeter, equal to 10 000 lbs. per square inch. An exception was made when building the previously mentioned bridge at Hamburg where the strain was raised to 9 kg. per square millimeter, or 12 800 lbs. per square inch. The weight saved by this means amounted to about 25 per cent. It is to be hoped that the regulations regarding strain will soon be

made general, and that a higher strain will be admitted for mild steel than that allowed at present for wrought iron.

Though mild steel does not possess any considerably higher ultimate tensile strength compared with wrought iron, yet the ratio of elasticity is much higher; it being in wrought iron about 16 kg., equal to 22 800 lbs., and in mild steel about 24 kg., equal to 34 100 lbs., an excess of 50 per cent.

As many civil engineers design with a fraction of ratio of elasticity, for instance with the half of it, the strain in wrought iron would be 7 kg., equal to 10 000 lbs., and in mild steel 10.5 kg., equal to 15 000 lbs. If it is not advisable to go so far, it will be at all events allowable to introduce a partial increase of the strain for the material, which may be fixed perhaps at 9 kg., or 12 800 lbs. Through this the question of cost would be favorably influenced.

Mild steel can be produced at a lower rate than wrought iron, the average price being at present 5% lower. Another advantage is that a greater strain may be allowed, which makes it in the end a decidedly more economical material. These combined advantages, together with higher qualities, will evidently secure the future of mild steel, and every extended application of such metal in general will advance its use. Not only are foundations by piles, grillages, sheet pilings of iron now made of it, but the framework for warehouses, etc., is beginning to be made of the same material, and the extension of iron building is going on in all departments.

It is to be hoped that no reaction will take place, and that those tendencies will not grow that incline toward the introduction of a harder kind of steel in order to obtain a higher limit of strain, and as a consequence a more economical construction.

Hopes are entertained that mild steel will be produced from iron and carbon only without containing other chemical substances, and that in this manner the quantity of carbon could be increased so far as to give to the material such strength that a higher strain can be employed. But the fact is overlooked that there still remains the possibility of hardening, with its dangerous consequences. Especially for bridge-building purposes this material would prove very unsuitable.

The custom in Germany, in constructing bridges of many pieces and connecting them with rivets, closely pitched, does not appear to be practicable for the use of a harder material. Another method of construction might be substituted, perhaps the application of eye-bars, as is often seen in America. Even then, the introduction of a harder steel would be of doubtful utility. It is therefore urgently desirable that the previously approved mild steel may still continue to be used for all constructions, without any alteration whatever.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

644.

(Vol. XXX.—November, 1893.)

THE PLANT OF MARITIME COMMERCIAL PORTS OF FRANCE.

By H. DESPRES, Ingénieur des Ponts et Chaussées.

Translated from the French by CHARLES WARREN HUNT, M. Am. Soc. C. E.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

Preliminary Observations.—The questions relating to the plant of maritime commercial ports, which, for a long time, were considered, at least in France, as being of relatively secondary importance, have for some years taken a commanding place in the attention of all those who are interested in the development and prosperity of maritime ports, and great efforts have been made on all sides to endow French ports with the means necessary to carry on their work in the most rational manner possible.

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

The object of this paper is to give a general review of the progress made in this direction, during the last 10 years, in the more recent installations. This rapid review will indicate the principles which, it seems to the writer, should guide in similar works.

It is first of all necessary to define what is meant by the plant of a commercial port. This name is given to the whole group of appliances which, while not indispensable to the existence of maritime traffic, facilitate its operations, render them more rapid and less onerous, or, in a word, ameliorate the conditions of work. In other terms, equipping a port is to get from it the greatest possible amount of essential work, from its basins and its quays, by fitting it out in the most profitable manner.

The capstans which, placed in the wing walls of the locks, facilitate and expedite the passage of vessels; the mechanisms which render the movements of gates and bridges more rapid; the machines which operate the discharge of merchandise; the sheds which shelter it during the loading and unloading; the warehouses where it accumulates; the special arrangements necessary to certain particular classes of commerce—these are some of the appliances which are to-day necessary to the life of a large maritime port and constitute its commercial plant. Perhaps the dry docks in which vessels can be repaired and rendered seaworthy should be added.

A double cause has retarded for a long time in France the complete organization of a perfect plant.

First of all, these individual parts of one whole do not originate in the same source and under one administration; the State Constructor of the Port usually furnishes it with dry docks, and always attends to the lifting engines on the locks; the railway companies construct the tracks on the quays; the Chambers of Commerce, ordinarily, build the sheds and put in motion the machinery for loading and unloading; separate organizations build the warehouses, and, in general, there is no one who can give attention to the special arrangements necessary to draw to a port a special commerce.

This diversity of the powers of many administrations which, nevertheless, should co-operate to one end, entails in practice many difficulties, and, in addition to this, the Constructor of the Port, who should have a primary interest in its proper working, has not the control over the plant which should belong to him. The plant is not only a facility

given to commerce, but it is an element of economy in the hands of the Constructor of the Port, which permits him to double and triple the usefulness of its quays and its terre-pleins, and, consequently, diminish the development of works which are so costly to establish.

Now, it must be recognized that the direct and palpable profit of the plant is not always very great, and that often it is impossible for the different parts which compose it to carry by themselves the expense of their own working, the payment of interest and the liquidation of their cost of construction, and this explains the hesitation of those interested in establishing them; but if this direct profit is sometimes debatable, the indirect benefit is always large, and even if the individual account of the sheds, the warehouses, or the machinery for loading and unloading should not show a profit, their establishment would, nevertheless, have procured to the port which possesses them many compensating advantages which would in the end justify the means.

To these difficulties, resulting from the conditions of administration operating the working of the ports of France, must be added those which are met with in the old ports, where, on the one hand, local habits of long standing have acquired considerable force, and have become difficult to modify, at least suddenly; or, on the other hand, the outlines of the works themselves are not well adapted to methods of work which were not foreseen at the time of their construction. This is the condition existing in the majority of French ports; it is forcible from many points of view, and has contributed not a little to increase the difficulties to be met from the special point of view which now occupies our attention.

To-day most of these difficulties have been overcome almost altogether by reason of the more intimate association which has arisen between the Chambers of Commerce and the State for the construction of new works, and under the spur of the most absolute need which rendered it necessary. It should not be supposed, to be sure, that there does not remain much to be done, and this the more as methods improve, and as needs continually change, but it is nevertheless true that a considerable step has been accomplished and that a glimpse can now be had of the approaching moment when French ports need not be envious in the matter of plant of the most favored among their competitors.

The following table gives the comparative situation, for the principal ports of France, of the development of the tracks, sheds and machinery for loading and unloading in 1884 and 1893, and shows the activity which has been displayed during this period in the improvement of the plant of these ports.

Ports.		Length of tracks on quays. Meters.	Area of sheds. Square meters.	Machines for loading and unloading.					
				Hand or steam.			Less than 3 tons.		
				100 tons and over.	20 to 100 tons.	3 to 20 tons.	Stationary steam.	Floating.	Hydraulic.
Marseilles..	1884.....	13 000	62 800	..	7	4	2	6	39
	1893.....	43 000	82 000	1	8	6	1	7	85
Havre.....	1884.....	20 000	54 000	1	2	8	5	4	..
	1893.....	37 000	108 000	1	3	9	12	9	44
Dunkirk. ..	1884.....	15 000	.....	..	..	..	..	..	..
	1893.....	35 000	20 500	..	2	1	19	9	(Projected) 21
Rouen.....	1884.....	8 500	.....	..	1	2	13	22	..
	1893.....	25 500	12 000	..	1	2	10	36	15
Bordeaux...	1884.....	14 500	.....	..	..	..	..	..	..
	1893.....	16 500	4 800	..	1	..	24	..	..
St. Nazaire.	1884.....	3 000	8 000	..	3	2	16	7	..
	1893.....	18 000	9 300	..	2	5	15	10	..
Dieppe.....	1884.....	6 000	2 400	..	..	..	..	..	..
	1893.....	10 000	5 300	..	1	4	29	..	..
Calais.....	1884.....	35 000	.....	..	..	3	8	..	..
	1893.....	21 000	23 000	..	1	5	8	..	16

Objects of the Plant.—The functions of commercial ports, of the large ones above all, are complex, and the changes made in commercial habits emphasize each succeeding day the double rôle they play. The growing facilities of communication, which multiply incessantly between the various portions of the globe, have as a natural consequence the drawing nearer together of producer and consumer and consequently the rapid development of transit.

It has become common to-day to say that commercial ports may, from this point of view, be considered simply as supplementary to internal ways of communication (railways and water-ways), and as an integral part of one large transfer station, where the sea-going vessel meets the cars or river craft, and where, even if the method of transport changes, the movement should not be stopped. But in addition to this rôle, the importance of which is increasing, the large ports add a second, not less essential because it affects the large com-

mercial markets like those of Havre or Marseilles; it is that of the bonded warehouse. Certain goods should in fact remain in place and undergo a warehousing, more or less long, waiting until they are sold or until again shipped, either by way of the harbor itself or toward the interior. Large markets, which are necessary to the life of commerce, will not be successful without an accumulation of goods, sometimes large, and the port then plays the part of a vast regulator, holding in its warehouses, at certain epochs, the production of far-off countries, in order to restore it during the rest of the year to the consumer.

This double part played by the ports indicates at once, as should be foreseen, that it is impossible to make general rules and to fix in a precise way what should constitute the plant of a port; it follows that when one of the two characters above indicated predominates, the installations themselves take a different character.

For direct transit operations the quays may be very narrow, discharging cranes and some lines of track, parallel to the quays, well connected with a distributing station, suffice; when the transit operates on goods which must be handled or have a preliminary examination, the terre-plein should become larger and the dépôt areas, either covered or not covered, should be on the same scale; if the goods handled must be bonded in the port, the installations should become more complete, and large warehouses, provided with all the necessary mechanisms, should be erected along the quay.

It seems that it would be at least easy in each port before it takes part, to a greater or less extent, in the divers kind of operations indicated, to specialize the quays, to prepare some for transit, others for storage.

While this is evidently the object which should be attempted, it is not always easy to make a complete distinction; first of all, the same vessels very often bring cargoes whose destination is very different, and such cargoes come more and more in the large cargo-boats, some of which carry to-day nearly 6 000 tons of freight.

In the second place, it is necessary to remember the fact that the number of new basins in each port capable of receiving the largest vessels is generally very limited, on account of the rapid increase in the dimensions of ships, and that it is, therefore, indispensable to arrange these basins in such a manner as to permit them to accomplish the different kinds of operations. It has also been the custom in France, in most of the basins constructed lately, to constitute a plant

which, while defective for transit and for warehouses, permits the quays to render at least the double service which may be demanded of them.

Railway Tracks.—The tracks are the most essential portion of the equipment of ports; the laying of them along the quays, while necessary for direct transit, is also useful for connecting them with the public warehouses and the industrial establishments. For the purpose of an active transit, three lines placed lengthwise along the quay are indispensable, these lines being moreover connected with each other by switches, at distances which do not exceed the length of a vessel, in such manner as to bring empty cars before each discharging vessel and to take away loaded cars without interfering with the operations of neighboring vessels. This plan is the one adopted at Dieppe on the coal dock, where it gives entire satisfaction (Plate I, Fig. 5).

On the new quays of the ports of Marseilles, Havre and Dunkirk, where the transit never reaches the same intensity, it is not necessary to resort to a similar plan.

The tracks at Bellot Basin, at the Port of Havre, are arranged in the following manner (Plate I, Fig. 1):

Along the north quay, of which the terre-plein is 89 m. wide, three groups each of two tracks are arranged. The first group, with track centers at 2.965 m. and 6.83 m. from the edge of the quay, is intended principally for transit operations; the one of those lines which is nearer to the quay is afterwards included between the two rails of a line of cranes; the two other groups are situated on either side of the service road, their center lines being at 57.785 m., and 62.785 m., and at 81.785 m., and 86.425 m. from the edge of the quay.

The first of these groups effects the loading and unloading of goods handled on the terre-pleins, and the second serves as siding and reserve.

Along the south quay, the width of which is 125 m., a similar arrangement of tracks is adopted, with this difference only, that the group of side and sorting tracks can be increased to five tracks as soon as the necessity is apparent.

All of these groups of lines are connected at frequent intervals, and the system has been worked out with the intention of permitting entire trains on all parts of the lines. The use of turn-tables has been absolutely prescribed, a sorting platform placed at the extremity of the basin is connected with the service lines of the quay by curves accessible to the largest rolling-stock in use on the French system, the

smallest radius allowed on the road being 125 m. For serving the short quays which separate the two tidal basins of the Bellot Basin, curves of 60 m. radius have been used, in order to avoid putting in turn-tables.

The arrangement of the lines of track of the Freycinet Basin at Dunkirk is somewhat analogous to that of the Bellot Basin, but the plan of the inclined tidal basins lends itself in a very happy way to a convenient operation by railroads (Plate I, Fig. 3).

The lines which operate each pier-wharf are five in number and include a group of two lines, the centers of which are at 3.25 m. and 10.75 m. from the edge; a second group of three lines is placed back of the sheds. The number of lines of these two groups can be made respectively three and five if that becomes necessary.

As all the tracks are connected by switches to the lines of the distributing station, approximately 140 meters are managed by right-angle connections, by means of turn-tables.

In the new wet dock of the Port of Calais (Plate I, Fig. 4) the system of tracks operating the quays has been considerably developed by the *Compagnie du Nord* with the hope of attracting to the quays of that port an important transit. On the West Quay two lines are placed along its edge, and a group of five lines is placed in the rear of the distributing platform of the freight depot. On the East Quay the number of tracks placed along the edge of the quay is now three. The two groups are connected from place to place by cross-overs furnished with turn-tables, but all their parts are, nevertheless, accessible to complete trains which travel only over curves of large radius.

The quays of the Port of Marseilles, which allow, also, a considerable development of tracks, are laid out on lines at right angles, which are less favorable to railway communication than those of the Freycinet Basin; the quays of the old basins (Arene and Lazaret) only communicating by turn-tables with the tracks of access to the maritime station. This plan, notwithstanding the use of hydraulic capstans, which, while lessening the inconvenience, are certainly less favorable to large freight movements, also without renouncing meanwhile the use of turn-tables (which in all the Mediterranean are regarded with a favor difficult to explain) has been arranged in such a manner as to connect by curves of large radius the tracks which operate the pier-wharfs of the Gare Maritime and National Basins with the maritime stations (Plate I, Fig. 9).

On the piers each quay is operated by three railways, one of which is placed near the edge of the quay, and the other two in the center of the pier, where there is, therefore, a group of four connected tracks. On the Pinède, which is reserved for mineral ores and coal, the number of tracks is five, divided into two groups, one of three tracks placed along the edge of the quay, and another of two tracks at some distance in the rear; these two groups, being connected by frequent right-angle communications provided with turn-tables.

The three important maritime and sorting docks, which at Marseilles are parallel to the general direction of the port, permit, as a consequence, the tracks of the quays to be easily served, and from this point of view, as from many others, the plant of this large port is worthy of the foremost French harbor.

The conditions at the Port of Rouen, where the quays spread out all along the Seine, and do not allow of basins, differ from those of the ports which we have just reviewed; the tracks which serve its quays extend along on each bank, not the least difficulty being encountered in their construction. On the north bank of the stream, three tracks, connected with groups of distributing tracks, run parallel to the quay, a single track is placed near the edge, and the other two in the rear of the terre-plein; on the south bank, two tracks run along the edge of the quay, and a single one is placed in the rear of the terre-plein.

If the development of these tracks is on a smaller scale than that of some of the ports before cited, it is due to the commanding place occupied at Rouen by the river boating, which performs the largest part of the transportation towards the interior.

Sheds.—In almost all the ports of France, the construction of sheds, intended to shelter goods during their stay on the terre-pleins, has been greatly developed; the regular lines of navigation receive in the interval between the times of departure merchandise for export, often valuable, which must sometimes be held, awaiting the departure of the next packet; for such goods the shed is an absolute necessity. While less necessary for imported cargoes, which can in general be shipped rapidly towards their destination (fluvial lighters, cars or warehouses), the sheds render, nevertheless, important service in accelerating the operations of unloading, which, by their use, are not interrupted by bad weather.

To render all the service for which they are intended, the sheds

should be at a short distance from the edge of the quay, and it should be easy to close them. In almost all French ports the distance from the face of the sheds to the edge of the quay is from 10 to 12 m., and provision for closing the sheds is everywhere made, except at Rouen, where the sheds play only the part of umbrellas.

At Havre the Chamber of Commerce has established a large number of sheds, which are much appreciated by commerce; the quays of the west dock of the Bellot Basin, in particular, have been furnished with sheds on almost all of their area (Plate I, Fig. 1).

On the North Quay three sheds 45 m. in width are built, overhang not included, the lengths of which are respectively 77.5, 139.5 and 93.0 m.; on the South Quay are also erected three sheds, of 55 m. width and of 80, 160 and 180 m. length. These sheds are separated from each other by open spaces of 40 m.; this break in the shelter is for the purpose of diminishing the chance of conflagration, to reserve on the quays parts of the terre-pleins for cumbersome merchandise and heavy weights, to reduce the expense of the installation by not erecting anything on the unused spaces which are found between two vessels, and, finally, to allow trucking to be easily carried on on the quay without traversing the sheds. The sheds have metallic frames with roofs of zinc plates, exterior walls of brick, all of the face situated on the side towards the quay being made up of movable doors (Plate III, Figs. 1 to 26).

The number of supports has been reduced as much as possible consistent with an economical construction, in order to present the least obstacle to the circulation of vehicles and to the piling up of goods; thus the sheds of 55 m. width are composed of trusses in 2 bays of 27.5 m. span, and 10 m. center to center. The principal dimensions of these buildings are summed up in the following table:

Sheds.	North. Meters.	South and West. Meters.
Span of rafters	22.50	27.50
Spacing of rafters	15.50	16.00
Total height of shed	11.64	12.60
Total height of doors	4.75	4.75
Number of intermediate rafters	2	....
Purlins	T iron.	Lattice.
Height of purlins	0.16	0.60
Least spacing of purlins	1.60	1.75

The net cost of the north sheds varies between 34 and 40 francs per square meter, not including the general charges for design and superintendence, which may be placed at 3 francs per square meter. The net cost of the south and west sheds, the spans of which are larger, has been nearly 42 francs, not including the general charges.

The cost is established by the square meter of covered surface between the sides of the sheds, that is to say, it does not include the surface covered by the overhang.

The net cost of the cheaper sheds can be itemized as follows:

		Francs.	Francs.
Superstructure.—	Foundations	.....	2.00
	Metallic frame	14.58	
	Roof	5.71	
	Battens	2.88	
	Gutters and leaders	0.88	
	Painting and glazing	0.27	
			24.32
Lateral Closing.—	Metallic frame	1.60	
	Wainscoting	1.51	
	Hardware for doors	1.23	
	Painting and glazing	0.48	
	Masonry	1.34	
			6.16
Miscellaneous.—	Drains	0.39	
	Paving of entrances	0.24	
	Lighting	0.23	
	Miscellaneous	0.66	
			1.52
	General charges		3.00
			37.00

The sheds built at Marseilles, Calais, Dunkirk and Dieppe are of about the same type as those adopted at Havre and present the same general arrangement.

At Marseilles the sheds of the Chamber of Commerce are 38 m. in width, divided into two equal spans; those of Calais are 40 m., equally divided into two spans; those of Dunkirk and of Dieppe have only one span, the length of which is 30 m. for the former and 24 m. for the latter. It is nevertheless necessary to describe the sheds which the Dock Company have recently established at Marseilles on the quays of the Lazaret Basin, and which carry a floor. The construction is made up

of trusses of 24 m. span of which the ridge is 13.05 m. above the ground. A floor placed at a height of 5 m. is supported by cast-iron columns spaced 6 m. center to center; an exterior gallery 3.60 m. wide runs at the level of the floor of the first story to receive goods which are deposited there by the hydraulic cranes (Plate II, Fig. 9). This arrangement, which has the inconvenience of incumbering, to some extent, the ground level with many supports, is, however, advantageous for narrow quays in that it doubles for them the surface which can be used for the storage of goods.

The shelters which the Chamber of Commerce of Rouen established some years ago differ from the general type on which the sheds have been constructed in the ports which have just been reviewed, in their mode of construction as well as in the absence of lateral closing which has been adopted in other ports. The roof is supported by small rafters of 5 m. span running perpendicularly to the quay and resting on longitudinal lattice girders 0.5 m. in depth; these girders are placed on plate supports 6 m. in height and spaced 10 m. center to center (Plate III, Fig. B5). All around the outside of the shed runs an overhang of 4.5 m. width. The sheds on the right bank thus cover a width of 26.2 m. including the overhang, and those on the left bank 34.2 m. including the overhang.

At Bordeaux, the Chamber of Commerce has erected on the quays of the Garonne pavilion-shelters of relatively small dimensions, 20 m. in depth by 30 m. in length. The rafters which support the roof have only 10 m. span and are 3.75 m., center to center. These shelters, of small dimensions, are useful in mild climates, since it is only necessary to shelter very costly merchandise. The following table indicates the net cost of the sheds at Havre, Calais, Dieppe, Rouen and Marseilles which were built at about the same time:

		Cost per Square Meter. Francs.
Marseilles.—	Sheds 38 m. wide	49
	" 25 m. "	50 to 57
Havre.—	Sheds 45 m. wide, with spans of 20.5 m.	37 to 40
	" 55 m. wide, with spans of 27.5 m.	45
Calais.—	Sheds 40 m. wide, with spans of 20 m.	43
Dieppe.—	Sheds 24 m. wide, with spans of 24 m.	43
Rouen.—	Sheds 34.2 m. and 26.2 m. wide, with spans of 25.2 and 17.2 m.	32

Tariffs of the Sheds.—The use of sheds and shelters established by the Chambers of Commerce on the quays of the various French ports allows the collection of a special tax designed to pay for the service rendered. In certain ports, such as Havre, Marseilles, etc., the taxes for use are the only resources which the Chambers of Commerce can place against working expenses, payment of interest and liquidation of the first cost. In others, such as Dunkirk and Rouen, the Chambers of Commerce collect in another way. A tax is levied on all ships which enter the port, whether they use any part of the plant or not. This tax, which is 0.1 franc at Dunkirk and 0.15 at Rouen, permits the reduction of the charges for use.

The determination of the method of levying these taxes has given rise to many differences, and there has been some trouble in finding a method which, while protecting all interests, satisfies in all respects the conditions of a rational rate, and also the exigencies of practice. A uniform method is not in use in the different ports.

For locations by the year made to regular companies the problem is easy, and it is sufficient to have a tax paid which is proportionate to the space occupied. The difficulty exists only for the establishment of an assessment for the use of sheds occupied by transient shipping.

At Marseilles and at Dieppe the tax is in proportion to the number of tons* of goods stored under the sheds. At Marseilles it is 0.5 franc per ton of 1 000 kg., and gives the right to a stay of 12 days under the shed, after which the tax becomes prohibitory and is calculated at the rate of 0.5 franc per ton per day during the first three days, and of 1 franc per ton for each following day. At Dieppe the tax is 0.08 franc per day per ton during the first three days and 0.09 franc per day per ton during the fourth, fifth and sixth, and 0.01 franc during the following days for goods of the first class (cereals, grain, coffees, sugars, cotton, wood); these charges are made respectively 0.1, 0.11 and 0.12 franc for other goods.

At Dunkirk and Calais the tax is also based on the quantity of goods stored, but it is computed from the chartered tonnage; it is at the rate of 0.06 franc per ton per day of the regulation stay, when this does not exceed five days, and 0.12 franc for each day after that for five days within the limit of the regulation period; 0.03 franc for each

* The ton used throughout this paper is the French *tonne* of 1 000 kg. or 2 204 lbs.—*Trans.*

day computed from the expiration of the regulation period during the first twenty days, and of 0.10 after the twentieth.

At Rouen the area of the sheds occupied regulates the tax, which is 0.05 franc per square meter during the first three days, and 0.15 franc during the three following days, and 0.2 franc after that.

At Havre the tax for occupation is based on the size of the vessel which is placed in front of the shed. The tax being otherwise variable in proportion to the width of the shed. It is paid by the vessel according to the tariff given below.

Size of shed.	Tax per day per net gauged ton.	
	Steamers.	Sailing vessels.
	Francs.	Francs.
Sheds wider than 45 m	0.09	0.075
" from 30 to 45 m. wide	0.08	0.065
" less than 30 m. wide	0.07	0.050

After the departure of the vessel the goods should be removed within a period of three days, after which a storage of 0.05 franc per day during the first five days, and 0.1 franc during the five following days, and 0.2 franc after 10 days, must be paid.

The mode of assessment adopted at Marseilles, Dieppe, Calais and Dunkirk has this great advantage: that the tax is in proportion to the service rendered, and each interested party knows in advance exactly the sum he will have to pay for the sheltering of his goods.

The method in use at Rouen is equally satisfactory from the point of view of the proportionality of the tax to the service rendered, and also that it incites those interested to occupy the least possible space with their goods. But it has one real inconvenience; those interested do not know in advance the sum they will have to pay, as this sum depends upon the manner in which the merchandise is stored under the sheds. It has an element of uncertainty which it would be better to avoid.

It should be stated that both methods of proceeding have a common inconvenience in the practical collection of the tax. To fix the amount it is necessary to make a measurement or a survey, sometimes long, whether it is to determine the volume and the weight of the goods, or the space occupied. It may also give rise to contest, and

this necessitates in all cases on the part of the agents of the Chamber of Commerce a work which may become very long and laborious if there are a great many claimants; it is to avoid these inconveniences that, at Havre, the gauged tonnage of vessels has been established as a common basis for the tax. The amount of the tax is thus easy to establish without possible contest and without any measurement whatever; the gauged tonnage being proportionate, at least very nearly, to the size of the vessel, the tax thus determined is sufficiently proportionate to the service rendered.

Machinery for Loading and Unloading.—The necessity of placing on the quays of ports machinery for loading and unloading is manifest. Sailing vessels do not possess windlasses which will allow them to discharge their goods economically, and the windlasses on which the steamers depend are not adequate to place on cars the goods which they can discharge upon the ground, and there are sometimes heavy loads which cannot be moved by the means on board.

These machines for loading and unloading, the number of which increases rapidly with the large demands of commerce, are of various kinds. On the one hand they can be fixed, movable on rails, or floating on the water; they can be worked by hand, steam, hydraulic power or electricity.

Hand cranes are to-day almost abandoned. There is no motive for their use except for tools of small power, the utilization of which is very rare. Fixed cranes, which oblige the vessel to move, are equally inconvenient and can only be allowed for extremely powerful engines, the first cost of which it is necessary to limit. The general desire is to have in the largest possible measure movable cranes of great power.

In all the large French ports there are to-day tools able to lift the heaviest load. At Marseilles there are nine engines which can lift packages of more than 20 tons; among them are found a stationary oscillating shears of 120 tons, worked by hydraulic power, and many pontoon steam derricks of which the strongest has a capacity of 60 tons. At Havre the number of machines of like power is more limited; there are only four, among which should be mentioned one stationary steam derrick of 100 tons and one floating oscillating derrick of 30 tons.

St. Nazaire, on account of its important ship-building yards, is equally well equipped from this point of view. There are found on its quays two fixed steam shears of 80 tons and one of 40 tons.

At Dunkirk the most powerful machine for discharging is one steam pontoon crane, pivoted, and of 40 tons power.

Machines of great power, nevertheless, are the exception, and most of those for loading and unloading are of a power varying between 1 000 and 2 000 kg., because almost all the packages which make up the ordinary cargoes are of a relatively small weight, and there is no advantage to be gained by assembling a large number of packages to make up each load.

Cranes, properly so called, which allow the placing of goods on the ground at a certain distance from the quay, or on cars, are the most useful and the most generally employed. Windlasses whose net cost is very much less are, however, also used, and in certain cases give good service.

The choice of the system of cranes to adopt for the plant of a port depends on very diverse and complex considerations. Floating pontoon cranes are very advantageous for ports where mechanical handling is little developed, and it is necessary to serve a long stretch of quays with a small number of machines. They offer advantages equally valuable when direct transfers from vessels to lighters are frequent, because they then allow the working of the ship on both sides ; their employment becomes on the contrary troublesome in ports where the plant has been well developed, and is also very difficult with large vessels, which are frequent to-day, in harbors subject to violent wind squalls, as the pontoons then suffer from being placed between the ship and the quay.

At Rouen, an interior harbor sheltered from heavy winds, and where fluvial transfers are of great importance, the use of floating cranes is easily justified and they form a most important part of the plant.

The floating-pontoon crane is therefore always an indispensable complement of all well-considered plants; there are always in fact in any port quays which are not furnished with stationary machines, and it is necessary that the lack of them should be supplied by means of a floating crane.

The movable crane on rails seems to be more generally employed, but the motors in use differ; up to the present time in practice, in France at least, only hydraulic and steam cranes are used ; many studies of electric cranes have been made, but it is not yet believed

necessary to return to them. The house of Bon & Lustremant has presented for the Port of Dunkirk a very interesting type of electric crane (Plate IV, Fig. 13), but for economical reasons the preference was given to a hydraulic plant.

The only example of the use of electricity for machines for loading and unloading is found at the docks of Havre, where small elevators run by this force are used to discharge grain in bulk.

The use of hydraulic power to actuate machines for loading and unloading is to-day very general and offers great advantages; all the apparatus of the plant is always found in working order at all hours of the day or night without preliminary kindling, the operation is simple and does not offer any danger of conflagration. Finally, from a single installation the cranes, capstans, gates or bridges can be operated. In cold countries some difficulty is found in working the cranes during the winter, but with trifling precautions, too numerous stoppages are, however, avoided.

Recourse has been had to hydraulic power in the recent installations at Calais, Dunkirk, Havre, Marseilles and Rouen. The movable steam crane is not altogether abandoned; it is necessary to resort to it in ports of less importance, which do not require a hydraulic plant, which is always costly, and also in ports operated by hydraulic machinery on such quays as are too distant from the machinery.

Hydraulic cranes are often placed on frames, in tunnel form, under which a railroad track can run, in order to economize space. This is the solution which has been adopted at Havre (Plate IV, Fig. 1). The only inconvenience which results is a slight additional expense for the construction of the crane, and a little greater difficulty in moving it.

At Havre the circulating crane track is composed of two rails placed respectively at 0.80 and 1.90 m. from the edge of the quay. The first of them, which is fixed on the granite, is a flat rail 0.1 m. wide and 0.03 m. thick, the corresponding crane wheel having double flanges; the second rail is placed on a longitudinal beam supported by cross-pieces, spaced 2 m. center to center. It has the form of a ⌐¬ iron 0.10 m. wide, 0.04 m. high; the face of this iron is at the level of the paving, so as not to be an obstacle to circulation; the corresponding wheel of the crane is an ordinary wheel without flanges.

The principal dimensions of the cranes are as follows:

Kind of Crane.	Speed of lifting loads.	Speed of orientation of crane.	Run of hook.	Height of center of block.	Reach of crane.	Angle of orientation.
	Meters.	Meters,	Meters.	Meters.	Meters.	Degrees
Hydraulic cranes of 1 250 kg	1.00	1.25	14.00	12	8.35	360
Hydraulic cranes of 3 000 kg	0.75	0.75	14.00	13	8.35	360
Steam cranes of 1 500 kg	1.00	1.25	16.00	12.60	11.60	180

The hydraulic canalization is composed of two conduits, one for water under pressure, the other for the return of water used; the pressure conduit is 0.1 m. in interior diameter, and the return conduit 0.125 m. The intakes are arranged in the rear of the edge of the quay; they are 12.50 m. distant center to center, and communicate with the general conduit by pipes of 0.02 m. diameter. The connection between these intakes and the lifting engines is made by means of flexible pipes which allow the cranes to be placed in any position whatever on the circulating track.

On the quays of the dock basin, the dock companies utilize, in preference to cranes, hydraulic windlasses; these machines, which are very easily handled, serve not only for the discharge of vessels, but for handling in the warehouses when the direct trans-shipment to cars is not made at the docks, and the parcels discharged are of a small enough weight, which condition is favorable to the exclusive employment of windlasses.

At Marseilles the Dock Company, as well as the Chamber of Commerce, has adopted hydraulic cranes for operating the quays equipped by them, to the almost complete exclusion of windlasses; the same is also the case at Calais and at Rouen.

The machines of these different ports are similar to those of Havre, and the type of frames in tunnel form is almost universal. This type has also been adopted at Havre for steam cranes on the coal quay for the same purpose of economizing the terre-plein.

What is the efficiency that it can be hoped to obtain from machinery for loading and unloading which has been placed on the quays of the ports, of which we have been speaking? This is a very important question; for as soon as these machines are established, a much higher

efficiency is expected than that attained in practice. The following table shows the efficiency based directly on the unloading in progress, the report being made without prejudiced anticipation in order to have results approaching the truth as nearly as possible :

Kind of Machine.	Goods.	Minimum Speed.				Maximum Speed.			
		Number of slings per hour.	Number of packages per sling.	Weight of package. Kilograms.	Tonnage discharged per hour. Tons.	Number of slings per hour.	Number of packages per sling.	Weight of packages. Kilograms.	Tonnage discharged per hour. Tons.
Hydraulic cranes of 1 250 kg. power	Cotton...	54	2	200	21.60	..	..	..	..
	Nickel ore	26	1	1 000 (Net.)	26.00	..	..	..	..
	Corn in sacks...	45	9	62	26.97	..	..	..	..
	Flour in sacks...	56	6	100	33.60	..	..	..	..
	Tobacco..	54	1	380	20.58	..	..	..	..
	Wine	38	2	550	41.80	..	..	..	..
	Mahogany logs.....	14	1	1 200	16.80	..	..	..	..
Hydraulic windlasses of 900 kg. power (Dock Company).....	Coffees. ..	45	10	60	27.00	65	10	60	39.00
	Corn in sacks...	38	1	65	25.00	55	10	65	36.00
Steam cranes { Rolling...	Coal.... .	32	1	1 000	32 00	48	1	900	43.00
Steam cranes { Floating..	Logwood .	48	1	300	14 40	..	..	..	..
Electric elevation for grain in bulk.....	Oleaginous grain	7 666	Buckets of 3 kg.		23.00	11 700	Buckets of 3 kg.		35.00
	Corn.....	7 500	" " 4 "		30.00	11 500	" " 4 "		46.00

The unloading can then be operated with the machines for that purpose in use at Havre, at the rate of 20 to 45 tons per hour per hatch, according to the kind of goods, which speed is satisfactory for the most urgent cases. Thus, a coal-boat having three hatches can expect to unload from 1 000 to 1 200 tons in a day of 10 hours' work. A large grain-boat discharging through four hatches can unload from 1 000 to 1 400 tons per day of 10 hours' work, and a cotton-boat with three hatches can unload 3 000 bales per day. For this last-mentioned goods, this result is not generally accomplished in practice, the unloading being retarded by the incumbering of the quay, due to the weighing and sampling of the bales.

The installation of machinery for loading and unloading on the quays is troublesome enough, and it is very rare, except in certain

particular establishments, that the return derived from the placing of these machines pays the costs of working as well as those of interest and the liquidation of capital. This it would certainly do if the use of the machines were continuous, but the fact is that a large number of vessels prefer to use the steam windlasses on board, and consequently the quay engines are often unemployed. In many ports the deficit in the account of the machinery for loading and unloading is covered by a general tonnage tax levied on navigation; in others, as, for instance, Havre, it is covered by the benefit derived from the use of the sheds.

The ordinary cranes are let generally by the hour or by the day, while the unloading of large packages is paid for by the ton. At Dunkirk and Dieppe all unloading is, however, paid for by weight.

For hydraulic cranes the tariffs in force are as follows:

Kind of Cranes.	Rent by the Day.			
	Marseilles. Francs.	Havre. Francs.	Rouen. Francs.	Calais. Francs.
Cranes of 1 250 kg.....	32 per day of 8 hours 40 " " 10 "	25	20	..
" 1 500 "	36 " " 8 " 45 " " 10 "	30	..	40
" 3 000 "	48 " " 8 " 60 " " 10 "	45	..	..
" 5 000 "	.....	..	..	70
" 10 000 "	.....	..	..	180
" 20 000 "	.....	..	..	190
" 40 000 "	.....	..	..	200
Shears of 120 tons..	240 per day of 8 hours 300 " " 10 "	..	..	..
Windlasses of 750 kg.....	20 " " 8 " 25 " " 10 "	18	..	25
" 1 000 "	.....	20	..	..

For the public steam cranes the charges per day are as follows:

Kind of Crane.	Rent by the Day.		
	Havre. Francs.	Rouen. Francs.	Bordeaux. Francs.
Cranes of 1 250 kg., Floating.....	30	30	..
" 1 500 " Rolling	50	..	50
" 1 500 " Floating	..	..	..
" 10 000 " Floating.....	100	..	..

The taxes levied on dischargements paid by the ton are as follows for packages of small weight:

	Rouen. Francs.	Dieppe. Francs.	Dunkirk. Francs.
Oils, minerals, etc	0.40	0.325	from 0.75
Other goods	0.60	0.575	to 1.25

For large packages the prices are as follows:

At Dunkirk the price per ton discharged varies from 3 to 9 francs when the packages weigh from 10 to 40 tons.

At Rouen it varies from 2 to 10 francs, as the weight of packages varies from 3 to 30 tons.

At Marseilles and at Havre the large number of machines of great power brings a competition, and the price varies according to circumstances and the law of supply and demand.

Warehouses and Storehouses.—The warehouses and storehouses which are found in French ports are handled by independent companies which are not connected with the administration of the port. These establishments are generally situated outside of the terre-pleins, and, in consequence are in a less favorable condition for use, a more or less troublesome transport being necessary to carry to them the goods discharged on the quays. The State does not concede the right for a long term to parts of the maritime domain except after lengthy formalities which often discourage the construction of warehouses on the quays.

Nevertheless, at the ports at Marseilles and Havre, there are dock companies, to which the State conceded certain lengths of quay, and which operate them. These companies have, since that time, been able to complete their storehouses under very favorable conditions on the very edge of the quays. In other ports, as, for instance, Bordeaux and Dieppe, the Chambers of Commerce have obtained in the same way the concession of certain parts of the terre-plein on which to erect storehouses.

The Dock Company at Marseilles, to which the State has conceded a length of quays of about 2 500 m., in the Joliette, Lazaret and Arene basins, was organized in 1854, and has remained the most important of the companies of this class in France. In 1892 these sheds and ware-

houses covered an area of more than 220 000 sq. m., which are distributed as follows:

Location.	Sheds. Sq. m.	Warehouses.			Total. Sq. m.
		Cellars. Sq. m.	Ground floor. Sq. m.	Upper floor. Sq. m.	
On the State Concession.....	27 600	.....	44 700	59 800	104 500
On the Company's land.....	.....	9 490	21 100	57 000	87 590
Total.....	27 600	9 490	65 800	116 800	192 090

For many years the Dock Company of Marseilles utilized water under pressure for all its loading and unloading, and its hydraulic plant was the first one in France. The company operates on the quay 53 hydraulic cranes, of which 17 have double power of 1 000 and 3 000 kg., 16 of 1 250 kg. power, and 22 of 1 000 kg. It possesses, besides, one steam crane of 20 tons and one floating elevator for the discharge of grain.

The traffic which is concentrated on the concession of the Dock Company of Marseilles is large, and its example shows well the benefit which a private company can derive from the operation of the quays conceded to it.

At Havre there is also a dock company, organized at about the same time as that of Marseilles, to which the State has conceded the possession of the dock basin, and of the south side of the Vauban Basin, that is to say, a length of 1 650 m. of quay for a period of 99 years.

The warehouses and establishments of the company are developed on the border of the quays, and have a capacity of from 150 000 to 180 000 tons of goods. In 1892 they covered the following area:

Sheds. Sq. m.	Covered Yards. Sq. m.	Warehouses.			
		Cellars. Sq. m.	Ground floor. Sq. m.	Upper floor. Sq. m.	Total Sq. m.
34 700	39 000	8 300	61 200	115 300	184 800

The plant of the company consists of 14 hydraulic windlasses of from 400 to 900 kg. capacity, used as much in discharging vessels as for handling in the storehouses, and of three grain elevators, with chains of buckets, operated by a generating dynamo worked by a 25 H. P. machine. These small movable elevators are easily placed on the point occupied by the vessels to be unloaded and permit their rapid and economical discharge.

At Havre, as at Marseilles and other ports, there are also many private storehouses, located far from the quays, to which they are connected by tracks. These warehouses are indicated in the following table:

	General Storehouses. Sq. m.	Pont Rouge Docks. Sq. m.	Raverat Storehouses. Sq. m.	Briquet Storehouses. Sq. m.
Ground floors.....	7 000	42 000	11 000	7 000
Upper floors	23 000	10 000	6 000	.....
Total	30 000	52 000	17 000	7 000

The Chambers of Commerce are beginning to try to give to commerce the advantages which appertain to docks established on the very border of the quays.

Thus, at Bordeaux, the Chamber of Commerce obtained in 1878 the concession for dock-warehouses occupying an area of 55 000 sq. m., at the very edge of the wet basin. The structures which it has already erected comprise one storehouse for wool, which covers an area of 13 500 sq. m., with cellars for storing wines and alcohol, and a large metallic building 380 m. long and 35 m. wide divided into two longitudinal parts, one used as a dépôt for goods which have settled the custom-house duty, the other for actual bonding. At Dieppe, the Chamber of Commerce has already received the concession of 6 000 sq. m. of ground near the new wet basin, in order to erect general storehouses, the operation of which has since been granted by the Chamber of Commerce to a private company.

Conclusion.—In the peaceful contest which all maritime nations maintain to-day to insure to their ports commercial supremacy, one of the indispensable weapons is the possession of a perfect plant, which,

Fig 1. PORT DU HAVRE

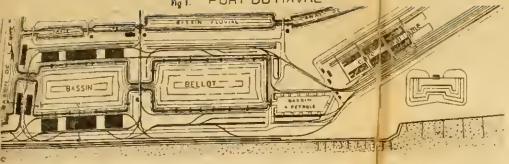


Fig 2. PORT DE ROUEN

Fig 3. PORT DE DUNKERQUE

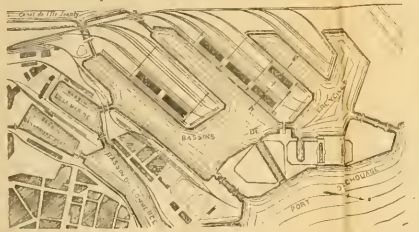
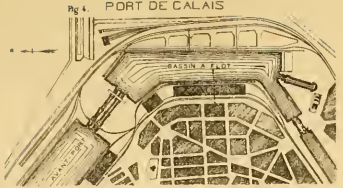


Fig 4. PORT DE CALAIS



PORT DE DIEPPE.

Fig 5.

Fig 6.

Fig 7. PORT DE St NAZAIRE

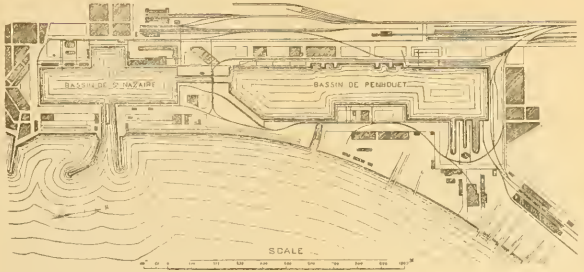


Fig 8 PORT DE BORDEAUX

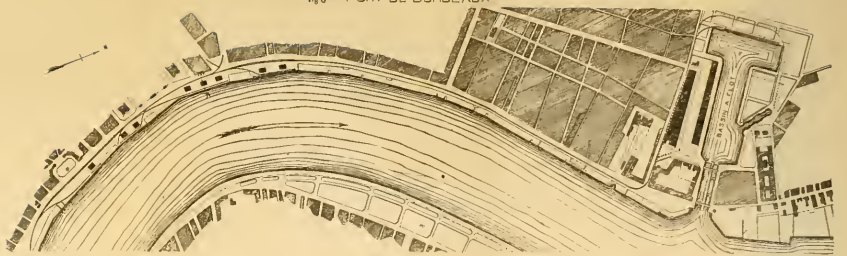
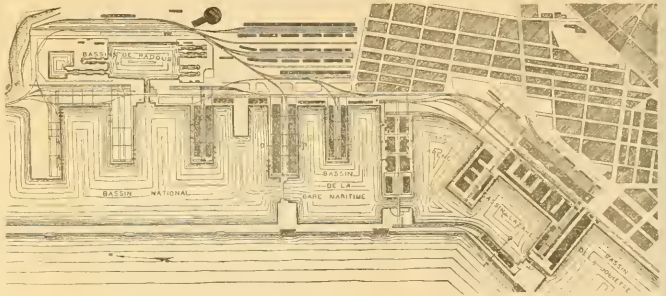


Fig 9. PORT DE MARSEILLE



VOL. XXX, No. 644.
TRANS. AM. SOC. CIV. ENGRS.
OESPRES ON THE PLANT OF FRENCH PORTS.
PLATE II.

PORT DE DUNKERQUE.

Fig. 1.

Coupe suivant GH.

PORT DE CALAIS

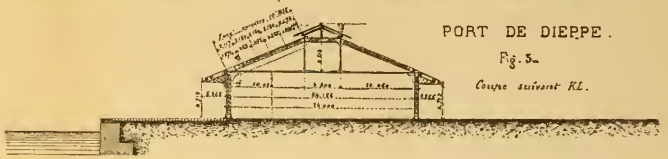
Fig. 2.

Coupe suivant IJ.

PORT DE DIEPPE.

Fig. 3.

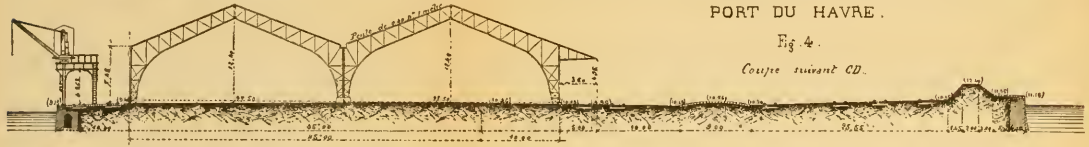
Coupe suivant KL.



PORT DU HAVRE.

Fig. 4.

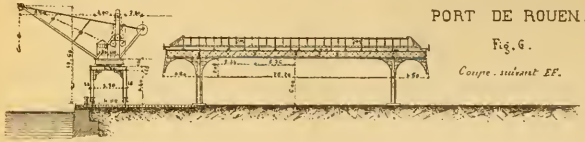
Coupe suivant CD.



PORT DE ROUEN.

Fig. 6.

Coupe suivant EF.



PORT DU HAVRE

Fig. 5.

Coupe suivant AB.

PORT DE BORDEAUX.

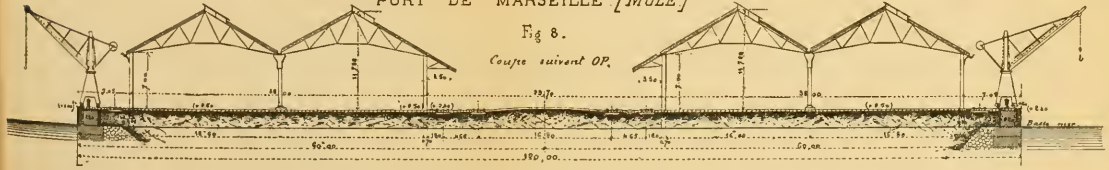
Fig. 7.

Coupe suivant MN.

PORT DE MARSEILLE *[MÔLE]*

Fig 8.

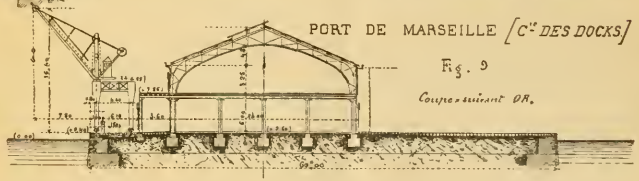
Coupe suivant OP.



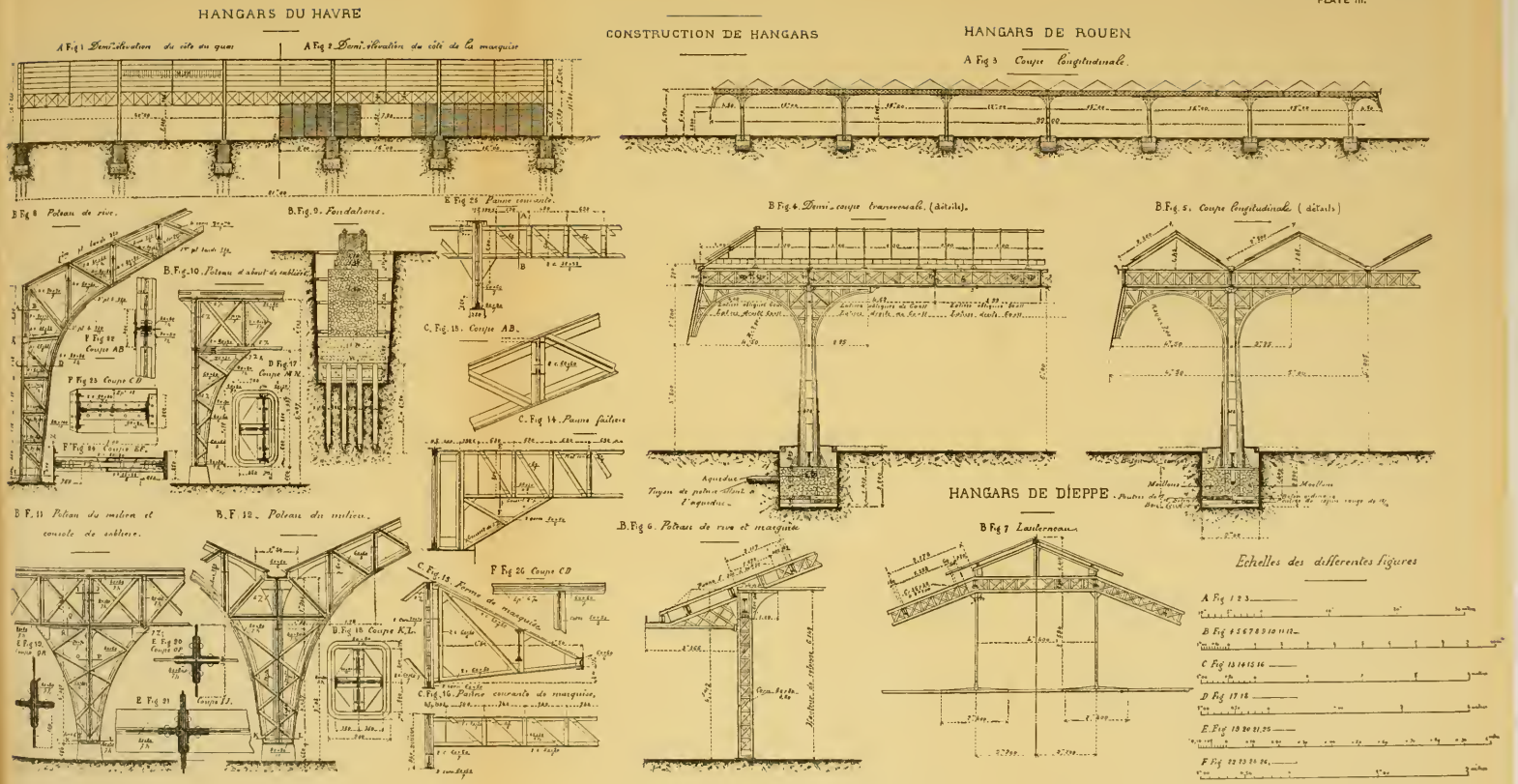
PORT DE MARSEILLE *[Cie DES DOCKS]*

Fig. 9

Coupe suivant QR.



VOL. XXX, No. 644.
TRANS. AM. SOC. CIV. ENGRS.
DESPRES ON THE PLANT OF FRENCH PORTS.
PLATE III.



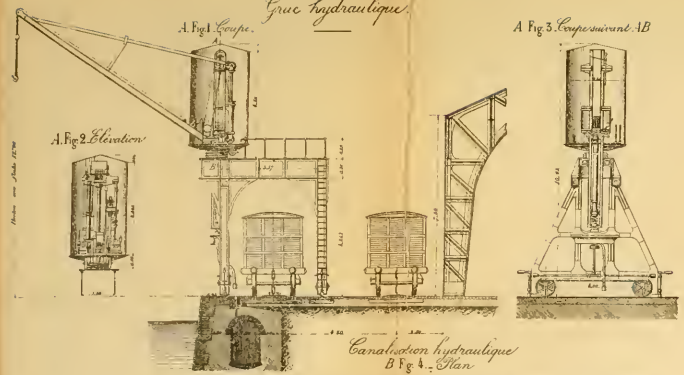
PORT DU HAVRE.

Grue hydraulique.

A Fig 1. Coupe.

A Fig 2. Elévation.

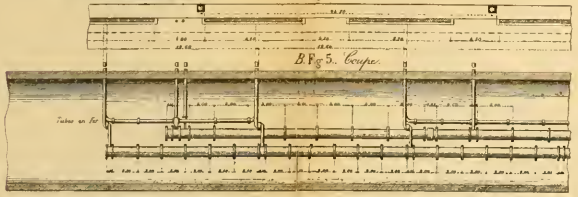
A Fig 3. Coupe suivant AB



Canalisation hydraulique

B Fig 4. Plan.

B Fig 5. Coupe.

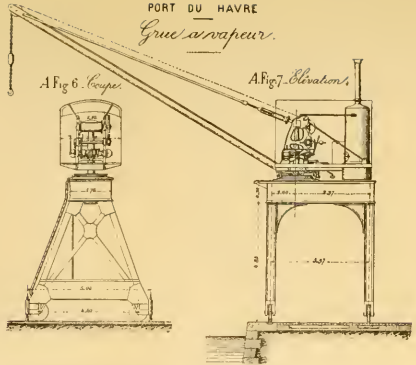


PORT DU HAVRE

Grue à vapeur.

A Fig 6. Coupe.

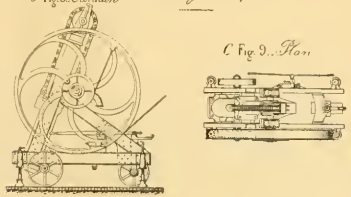
A Fig 7. Elévation.



Treuil hydraulique

C Fig 8. Elévation

C Fig 9. Plan.

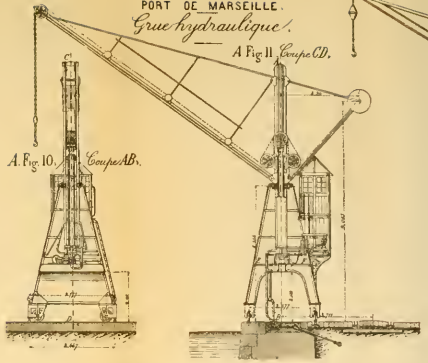


PORT DE MARSEILLE.

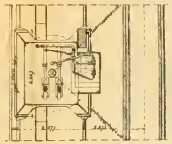
Grue hydraulique.

A Fig 10. Coupe AB.

A Fig 11. Coupe CD.

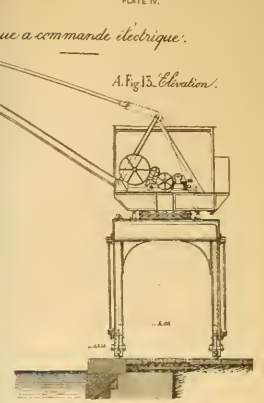


A Fig 12. Coupe suivant EFGH.

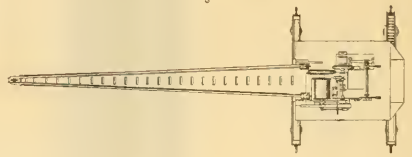


Grue à commande électrique.

A Fig 13. Elévation.



A Fig 14. Plan.



A Scale A.

Scale B

Scale C

in reducing as much as possible unnecessary cost and loss of time, promotes the greatest usefulness of these ports.

The works undertaken in France, from this point of view, during the last 10 years, and to which we have given a very rapid glance, have transformed the conditions of the work of commercial ports, at the same time that the large development of water-ways and railways has completed the system of communications with the interior of the country. The endeavor has also been during this period to give primary attention to the general necessities of commerce and to furnish quays which can respond to almost all needs. It remains, perhaps, now to determine whether it is not necessary to specialize certain parts of ports, and to give to them arrangements appropriate to particular needs; but everything cannot be undertaken at once, and the progress accomplished is a certain gauge by which the near future will be able to realize what remains to be done.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

645.

(Vol. XXX.—November, 1893.)

THE DEVELOPMENT OF QUAY-CRANES IN THE PORT OF HAMBURG.

By CHR. NEHLS, Chief Engineer to River and Harbor Board of Hamburg, Germany.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

The City of Hamburg is situated 100 km. (62 miles) from the mouth of the River Elbe. Its harbor is accessible to ocean-going vessels, but from the harbor upwards only river vessels are employed. The mouth of the Elbe is subject to the tides. The flood tide in Hamburg rises to the height of nearly 2 m. (6½ ft.). Vessels with a draught of 7.2 m. (23 ft. 6 ins.) can go with the tide to Hamburg Harbor, under normal conditions, without discharging their cargo.

In the year 1891, the imports amounted to 8 900 000 tons, and the exports to 5 300 000 tons. A considerable amount of goods, 3 100 000 tons, was discharged from 3 758 ships at the quays, which extend over

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

a length of 10.3 km. ($6\frac{1}{3}$ miles). On these quays the following cranes were working :

A.—Stationary Cranes.

1	steam crane, of.....	150 000 kg.	lifting power.
1	" "	50 000 "	"
1	" "	12 500 "	"
1	hydraulic crane, of	5 000 "	"
17	" cranes, each of..	2 000 "	"
4	" " " ..	1 500 "	"
4	" lifts, " ..	1 000 "	"
10	gas cranes, " ..	400 "	"
1	hand crane, " ..	40 000 "	"
21	" cranes, " ..	2 500 "	"
1	" crane, " ..	1 500 "	"
4	" cranes, " ..	1 000 "	"

B.—Traveling Cranes.

1	steam crane, of.....	7 500 kg.	lifting power.
3	" cranes, each of.....	5 000 "	"
1	" crane, of.....	3 000 "	"
125	" cranes, each of.....	2 500 "	"
123	" " "	1 500 "	"
2	electric " "	2 500 "	"
6	hand " "	2 500 "	"
94	" " "	1 000 "	"

Making a total of 421 cranes, with a collective lifting power of 1 000 000 kg., which is more or less uniformly distributed over the quays, and gives an average lifting power equal to 96 kg. per meter, or 30 kg. per foot, of quay length. Hamburg stands unrivaled in this respect.

A little more than a quarter of a century ago there were no quays and only a very few primitive cranes. Vessels to be loaded or discharged were moored to strong sets of piles driven into the bed of the river, called Duc d'Alben, and discharged and loaded from lighters by means of hand cranes. These lighters carried the cargo to the warehouses, which were built on the banks of canals and branches of the Elbe. These light-draught vessels could load and discharge alongside the warehouses.

But with the advance of railway enterprise and the great development of steam navigation a new carrying trade was called into existence which was unknown before. Part of the arriving goods were not taken to the storehouses, but were used as transit goods and carried by the railways directly to the steamers, and *vice versa*. Then, again, the size of the steamers was constantly increasing. The vessels took cargo for different receivers, and the goods had therefore to be sorted on arrival. The consequence of the great revival of commerce was that steamers had to make quicker voyages. Loading and discharging had therefore to be accelerated, and consequently efficient cranes had to be built. Under these circumstances the building of quays was begun for steamers in Hamburg, the first one being the Sandthor quai which was opened in the year 1866 (Plates 1 and 2).

It was a difficult task to construct the cranes so as to meet all requirements.

First of all, there was no doubt that traveling cranes only could be used, with the exception of cases where heavy loads had to be lifted. The necessity of discharging simultaneously from all the hatches, and the different position of these according to the length and construction of the steamers, was the reason why traveling cranes were deemed to be most practicable.

The variable beam of steamers, and the circumstance that at times lighters or river vessels have to be brought between the vessel and the quay, made it desirable to work the cranes with variable projecting jibs. The smallest projection was fixed at 6.8 m. (22½ ft.), the largest 9.1 m. (30 ft.), and the corresponding lifting power had to be 1 600 kg. in the former, and 1 200 kg. in the latter case. With the small jib goods can be lifted out of the hold of the ship and deposited in the sheds which run parallel to the quay, just behind the railway lines, the goods being carried clear of the railway lines lying between the crane rails and the shed. The greatest projection of jib is sufficient when a lighter of greatest beam is placed between the quay and steamer.

The highest point of the cranes could not be more than 6 m. (20 ft.) above the top of the rails, in order that the crane might swing round easily below the roofs of the sheds. The maximum lift was to be 10.3 m. (34 ft.), to allow the working of the cranes under all circumstances and at any height of the river. The velocity of lifting was fixed at 0.6

m. (2 ft.) per second. These were the conditions laid down for the design of the cranes. Several designs were sent in, the motive power being air, water and steam.

The offer of Brown Brothers, in Edinburgh, was recognized as the most suitable for the purpose, and this firm secured the order for 16 cranes. The motive power in their design was steam. The crane is placed upon a bogie, with a gauge of 2.36 m. (7 ft. 9 ins.), which carries in the middle the crane post.

By means of the piston in the cylinder C, the crane revolves upon the pin, and is guided by three wheels which run on a collar of the sole-plate (see Fig. 1, showing the steam cranes as constructed and improved in many respects in later years, as well as the unaltered principle of the construction).

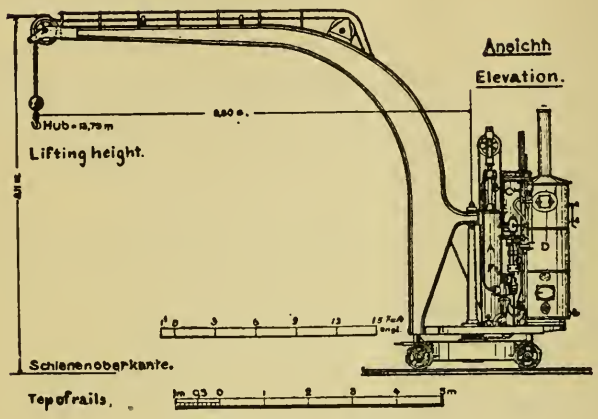
The crane is so balanced by the boiler that when working without load the two after guide wheels touch the guide ring; whereas, when loaded, the forward and larger wheel comes in contact with the guide ring.

To the movable platform are attached two vertical steam cylinders of 400 mm. (16 ins.) diameter and 1 830 mm. (6 ft.) stroke. The two piston-rods are connected by means of a cross-head which carries three movable pulleys of a set; whereas, the three fixed pulleys are placed between the cylinders. A chain is taken round these six pulleys, the end of which is carried over the top pulley of the jib. As soon as steam enters below the piston, the set of pulleys is forced outwards, and the load is lifted with a speed six times greater than that of the piston.

Between the two cylinders is fixed a hydraulic cylinder B, whose piston-rod is also connected with the cross-head so as to take part in the motion of the steam piston. This cylinder is used for the brake when the steam pistons ascend, thereby lifting the load; the water above the piston of the third cylinder is forced into a receiver. In this way the water passes an automatic valve, which is partially shut as soon as the flowing water exceeds a certain speed. For instance, in case the chain should break, the speed of the steam pistons is accelerated, and these draw with them the piston of the hydraulic cylinder. But the latter follows slowly, as the automatic valve is partially shut and the steam pistons are checked, and damage to the crane is prevented.

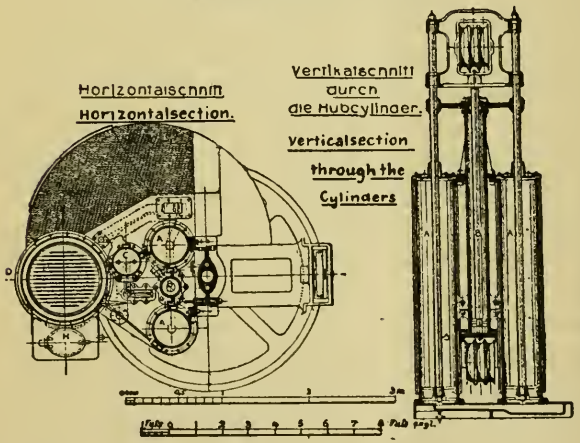
Having exhausted the steam, the pistons move downwards when the

FIG. 1.



Fahrbarer Dampfkrahn.

Moveable Steamcrane.



Fahrbarer Dampfkrahn.

A, steam cylinder. *B*, hydraulic cylinder. *C*, cylinder for swinging gear. *D*, boiler for 10 atmospheres with 2 500 kg., for 7.5 atmospheres with 1 500 kg. lifting power. *E*, starting lever. *F*, hand lever for the hydraulic valve. *G*, hand lever for the cylinder for swinging gear. *H*, water tank.

load is to be put down, and the water in the hydraulic cylinder below the piston is forced through another pipe into the receiver. In this way the water also passes through the valve, which is controlled by the crane man by means of a lever. According to the opening of the valve the loads can be put down quickly or slowly. The crane is worked by one man.

Steam is generated in the boiler D, having a combustion chamber with cross-tube of 340 mm. (13½ ins.) diameter. The exhaust steam is conducted to a heater, where the feed water is heated to about 25° Cent., and then forced into the boiler by means of a steam feed pump. The crane can be turned in the 0.55 part of a circle.

The forward part of the jib is made on the telescopic principle, and is worked by means of a chain which is taken round a shaft. In order to prevent any strain coming on the chain, holes are bored a foot apart, and a bolt is put in these which takes up the pressure from the chain. The maximum projection of the jib is 9.7 m. (31 ft. 6 ins.), but it has often happened that the jib extended 10 m. (33 ft.) when it was lowered, and it was possible to discharge from a lighter of 4 m. (14 ft.) beam between the ship and the quay into the hatch of the vessel.

The cranes are moved along the rails by manual labor, but, in order to bring them to their exact position, ratchet gear is fitted, a spur wheel turning the wheels of the bogie. Figure 1 shows this arrangement.

The following is the result of the trials which were instituted, to test the efficiency of the cranes.

The average number of kilograms of goods moved per hour was 26 150, of which one-half was lifted out of the hold on to the quay and the other half lifted from the quay and deposited in the lighters. The average lifting height was 5.15 m. (18 ft.). The average coal consumption was 9.75 kg. per 5 000 kg. of goods moved, but it must be borne in mind that the goods were slung and deposited without stoppages. The crane worked equally rapidly with large and small jib projection, and the weight of the load had no influence whatever upon the velocity of lifting. The crane worked almost noiselessly. The quantity of steam used was remarkably small, but this was due to the admission of a quantity of steam into the cylinder proportionate to the weight of the load and then cutting off the supply. From this it will be clear that the man working the crane must

be a skillful and experienced engineer. The steam pressure varied between 6 and 8 atm. (90 and 120 lbs.). When working in the ordinary way the coal consumption was larger, as there were many pauses when the cranes were not used, but steam had to be kept up.

Besides these steam cranes, hand cranes were erected which have a jib projection of 6 m. (20 ft.). They are for use in lowering and depositing goods out of the sheds into the lighters. In this case the lifting height is very small; they can easily be swung clear of the sheds, and the load let down into the lighters by its own weight. It would be too expensive to use steam cranes for this work.

Part of these hand cranes were of the ordinary design, and part of them were hydraulic cranes. In the latter case the crane post was hollow and used as a hydraulic cylinder. Its piston was connected with a set of pulleys, and the chain to these pulleys. The water was pumped into this cylinder out of an air-vessel. The water flowed into this air-vessel when the load descended, and compressed the air contained in the vessel. A set of pulleys lifted a balance weight when the load was lowered, and the weight of this balance weight, together with the pressure of the compressed air, caused the chain to ascend again automatically. These cranes, however, were very difficult to manage, and it was decided not to order any more.

Two movable steam cranes for heavy work were then erected, capable of lifting 3 000 and 5 000 kg., respectively. These are wheel-geared cranes. The engine turns a shaft, and this turns the other wheels by means of conical friction gear. The lifting, lowering and swinging of the cranes is done by the engine, which in this case is non-reversible, and therefore they work very rapidly. If the load of the heavier crane is greater than 2 500 kg., it has to be fitted with a balance weight which is always kept ready for use.

The following 27 cranes were the first erected:

1 steam-crane of 5 000 kg. lifting power.
1 " " 3 000 " "
17 steam-cranes " 1 500 " " each.
4 hydraulic hand-cranes of 1 000 kg. lifting power each.
4 ordinary " " 1 000 " " "

When the quays were extended the number of cranes had to be increased. Table No. 1 shows this clearly. It should be mentioned that

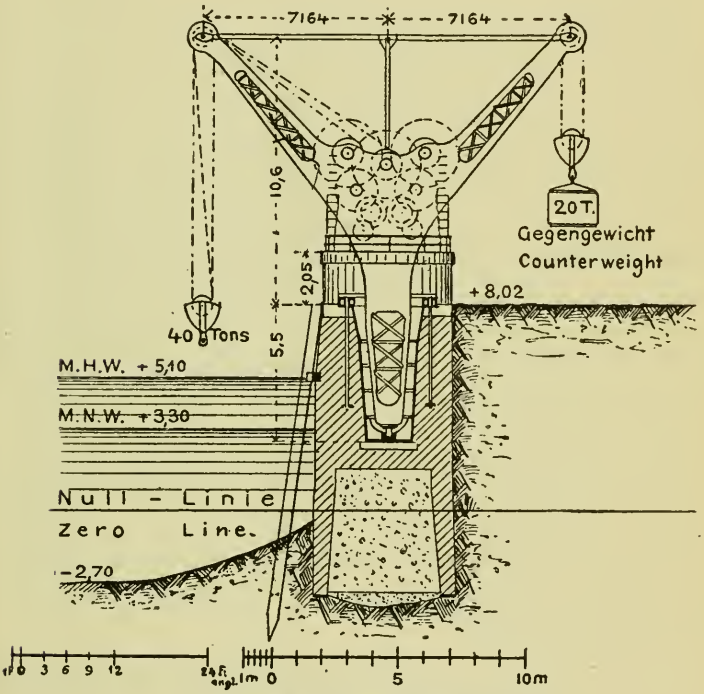
TABLE No. 1.

No.	When built.	Position.	Number of cranes. Fixed.	Number of cranes. Movable.	Lifting power. Kg.	Projection. Center of pin to chain. Feet.	Projection. Center of pin to chain. M.	Projection. Edge of quay to chain. Feet.	Projection. Edge of quay to chain. M.	Lifting height. Feet.	Lifting height. M.	Motor.	Principle of construction.	Arrangement of brake.	Remarks.
1.	1866.	Sandthor quay	.....	17	1 500	42° 3"	6.80	17° 6"	5.17						
2.		" "	.....	6	1 000	19° 8"	6.00	14° 4"	4.37						
3.		" "	.....	1	3 000	22° 6"	6.86	17° 2"	5.23	.....	.....	Hydraulic power			
3		" "	.....	1	6 000	23° 7"	7.20	18° 0"	5.57	.....	.....	" "			Lifts.
4	1867.	Kaiser quay	.....	24	1 500	29° 10" or	9.1 or	24° 3" or	7.4 or	.....	.....	Hand			
5	1872.	Dalmann quay	.....	6	2 500	25° 11"	7.9	20° 4"	6.2	.....	.....	Gas			
6.		Kaiser quay	1	.....	12 500	23° 6"	7.16	15° 5"	4.70	.....	.....	Hand			
7.		Dalmann quay	1	.....	40 000	23° 6"	7.16	15° 2"	4.64	.....	.....	Hydraulic power			Near the water.
8.		Kaiser & Dalmann quay	.....	10	1 000	19° 8"	6.00	14° 1"	4.29	.....	.....	" "			In the shed on the rails
9.	1875.	Lengthened Sandthor quay	.....	4	1 500	26° 5"	7.74	20° 0"	6.11	.....	.....	" "			In the street.
10.		" "	.....	4	2 500	26° 5"	7.74	20° 0"	6.11	.....	.....	" "			Near the sand pier.
10.		" "	.....	1	5 000	26° 11"	8.20	21° 7"	6.57	.....	.....	Hand			
11.		" "	.....	1	1 000	19° 8"	6.00	14° 4"	4.37						
12.			.....	12	1 500	32° 2"	9.80	26° 7"	8.10						
13.	1876.	Dalmann quay	.....	11	1 000	21° 6"	6.55	15° 11"	4.65						
14.		" "	.....	6	1 500	32° 2"	9.80	26° 7"	8.10						
15.	1877.	Hübener quay	.....	6	1 500	25° 5"	7.75	19° 10"	6.05						
16.		For distribution	.....	3	1 500	21° 6"	6.55	15° 11"	4.85						
17.		Kuleuer quay	.....	4	1 000	21° 6"	6.55	15° 11"	4.85						
18.	1878.	For distribution	.....	5	1 500	25° 6"	7.75	19° 10"	6.05						
19.		" "	.....	1	2 500	25° 5"	7.75	19° 10"	6.05						
20.		" "	.....	1	1 500	25° 5"	7.75	19° 10"	6.05						
21.		Straud quay	.....	5	1 000	21° 6"	6.55	15° 11"	4.85						
22.	1880.	" "	.....	1	2 500	26° 3"	8.00	20° 0"	6.30						
23.		"	.....	10	1 500	25° 6" to 32° 2"	7.75 to 9.80	19° 10" to 26° 7"	6.05 to 8.10						Substitute for older cranes
24.	1882.	Kaiser quay	.....	1	7 500	27° 0"	8.23	21° 5"	6.52						
25.	1884.	For distribution	.....	8	1 500	25° 6"	7.75	19° 10"	6.05						
26.		" "	.....	7	2 500	26° 3"	6.00	20° 8"	6.30						
27.	1885.	" "	.....	2	1 500	32° 2"	9.80	26° 7"	8.10						
28.		" "	.....	2	2 500	25° 5"	7.75	19° 10"	6.05						
29.		Sandthor quay	3	.....		7° 7"	2.32	4° 7"	1.40*						
		For distribution	7	.....	2 500	9° 0"	2.74	5° 3"	1.60*						
30.	1886-88	For distribution	.....	2	2 500	32° 2"	9.80	26° 7"	8.10	45° 1"	13.75	Steam	Blocks	Hydraulic brake	Substitute for older cranes
31			.....	7	1 500	25° 5"	7.75	19° 10"	6.05	36° 1"	11.00	"	"	"	
32.	1889	Versmann quay	.....	25	1 500	32° 2"	9.80	26° 6"	8.07	45° 1"	13.75	" {112½ lbs. {7½ atm.	"	"	
33		Amerika quay	.....	12	2 500	32° 2"	9.80	26° 6"	8.07	45° 1"	13.75	" {150 lbs. {10 atm.	"	"	
34		Krahnhoft	1	.....	150 000	56° 9"	17.30	32° 10"	10.00	.....	.....	" {75 lbs. {6 atm.			
35.		Versmann quay	.....	15	1 000	29° 6"	9.00	23° 10"	7.27	33° 7"	10.25	Hand	Wheel gearing.	Centrifugal brake.	
36.		Amerika quay	.....	8	1 000	29° 6"	9 00	23° 10"	7.27	33° 7"	10.25	"	"	"	
37.		Versmann and Amerika quay	.....	6	2 500	10° 8"	3 25	5° 10"	1.79*	25° 0"	7.60	"	"	"	Wall cranes.
38.	1889-90.	Petersen quay	.....	32	2 500	36° 1"	11.00	27° 1"	8 25	45° 1"	13.75	Steam, {195 lbs. {13 atm.	Blocks	Hydraulic brake	Semi-arched.
39.		Asia quay	.....	27	2 500	36° 1"	11.00	27° 1"	8 25	59° 0"	18.00	" "	"	"	"
40.	1889-90.	Krahnhoft	.....	4	1 500	27° 11"	8.50	22° 3"	6.77	45° 1"	13.75	Steam	"	"	
41.		Amerika quay	.....	4	1 500	32° 2"	9.80	26° 6"	8.07	45° 1"	13.75	"	"	"	
42.		For distribution	.....	2	2 500	25° 5"	7.76	19° 9"	6.02	36° 1"	11.00	"	"	"	Partly substitutes.
43.		"	.....	5	2 500	26° 3"	8.00	20° 7"	6.27	45° 1"	13.75	"	"	"	
44.		"	.....	4	2 500	32° 2"	9.80	26° 6"	8.07	45° 1"	13.75	"	"	"	
45.		Petersen quay	.....	12	1 000	33° 9"	10.30	24° 11"	7.30	45° 1"	13.75	Hand	Wheel gearing.	Centrifugal brake	Semi-arched.
46.		Asia quay	.....	9	1 000	33° 9"	10.30	25° 11"	7.30	45° 1"	13.75	"	"	"	
47.		Petersen quay	.....	12	1 000	11° 6"	3.50	19° 4"	5.99*	16° 5"	5.00	"	"	"	Semi-arched at the back of the sheds.
48.		Asia quay	10	.....	2 500	14° 1"	4.30	9° 1"	2.77*	.....	.....	"	Wiech		Wall cranes
49.		Amerika quay	.....	4	1 000	29° 6"	9.00	23° 10"	7.27	33° 7"	10.25	"	Wheel gearing.	Centrifugal brake	Substitutes.
50.	1891.	Kirchenpauer quay	.....	9	2 500	32° 2"	9.80	26° 4"	8.03	45° 1"	13.75	Steam	Blocks	Hydraulic brake	
51.		"	.....	4	1 000	11° 6"	3 50	19° 4"	5.90*	16° 5"	5.00	Hand	Wheel gearing.	Centrifugal brake	Semi-arched at the back of the sheds.
52.		Petersen quay	.....	2	2 500	36° 1"	11.00	27° 1"	8.25	45° 1"	13.75	Electricity	.....	.....	Semi-arched.
53.		For distribution	.....	13	2 500	2° 9" to 32° 2"	7° 25" to 9 80	18° 1" to 26° 4"	6.52 to 8.03	36° 1" to 45° 1"	11.0 to 13.75	Steam, {150 lbs. {10 atm.	Blocks	Hydraulic brake	Partly substitutes.
54		Asia quay	.....	2	2 500	36° 1"	11.00	27° 1"	8.25	59° 0"	18.00	" {150 lbs. {10 atm.	"	"	Semi-arched.
55.		Amerika quay	.....	1	5 000	31° 2"	9.50	25° 6"	7.77	.....	.....	Steam	.....	Band brake and centrifugal brake	
56		Baakenhoft	1	.....	50 000	41° 10"	12.75	29° 2"	8.90	59° 0"	18.00	"	.....	Weston's brake.	
		Besides these there are the following lifting arrangements.	24	374	.....	.....	.....	.....	.....	39° 10"	10.30	" {90–120 lbs. {6–8 atm.	Blocks	Hydraulic brake	
												Hand	Wheel gearing.	Band brake	Four of these cranes worked with the help of hydraulic power.
		Quaispeicher "A"	4	.....	1 500	.....	.....	.....	.....	.....	.....	Steam	"		
		" "	4	.....	1 000	.....	.....	.....	.....	.....	.....	"	"		
		" "	4	.....	1 000	.....	.....	.....	.....	.....	.....	"	Blocks	Hydraulic brake.	
		" "B"	10	.....	400	.....	.....	.....	.....	.....	.....	"	"	"	
		" "	1	.....	2 500	.....	.....	.....	.....	.....	.....	"	Wheel gearing.		
		Magdeburger quay	9	.....	2 000	16° 6"	5.00	12° 6"	3.60	.....	.....	Hand	"		
		" "	6	.....	2 000	11° 6"	3.50	6° 6"	1.95*	.....	.....	"	"	Band brake.	
		" "	2	.....	2 000	9° 10"	3.00	4° 11"	1.60*	35° 9"	10.90	Steam, {110 lbs. {7½ atm.	Blocks	Hydraulic brake	
		Brooksfleth	1	.....	5 000	18° 0"	5.50	13° 2"	4.00	35° 9"	10.90	" {150 lbs. {88 atm	"	"	
		Wilhelmineu platz	1	.....	1 500	.....	.....	.....	.....	.....	.....	" {67½ lbs. {4½ atm.	Wheel gearing.		
		Total	66	374						34° 1"	10.40	Hand	"		
										45° 1"	13.75	Steam	Blocks	Hydraulic brake.	
										33° 7"	10.25	Hand	Wheel gearing.	Band brake.	
										45° 1"	13.75	Steam	Blocks	Hydraulic brake.	
										35° 9"	10.90	"	"	"	Substitute for older cranes.
										33° 7"	10.25	Hand	Wheel gearing.	Band brake.	
										35° 9"	10.90	Steam	Blocks	Hydraulic brake	
										45° 1"	13.75	" {150 lbs. {10 atm.	"	"	
										35° 9"	10.90	Steam	"	Friction brake.	
										33° 7"	10.25	Hand	Wheel gearing.	Band brake.	
										45° 1"	13.75	Steam, {150 lbs. {10 atm.	Blocks	Hydraulic brake	Partial substitute for older cranes.
										36° 1" to 45° 1"	11.0 to 13.75	" {112½ lbs. {7½ atm.	"	"	
												"	Wheel gearing.	Band brake.	
										36° 1"	11.00	" {112½ lbs {7½ atm.	Blocks	Hydraulic brake.	
										45° 1"	13.75	" {150 lbs. {10 atm.	"	"	
										45° 1"	13.75	"	"	"	
										36° 1"	11.00	"	"	"	
												Hand	Wheel gearing.	Band brake.	Wall cranes at the back of the sheds.

* Edge of platform to chain.

with the cranes of No. 5, and also those erected at a later period, the variable projecting jib was done away with, as it did not prove successful. Some of the cranes were fitted with curved jibs, in order to clear the high bulwarks of large steamers and also the railway tracks. These curved jibs are more flexible than the straight ones, and better fitted to stand the shocks to which the load on the chain is exposed. This system of jib, which proved a success, has since been adopted in all steam-cranes.

FIG. 2



Handkrahn am Dalmannquai.

Handcrane on the Dalmannquai.

The crane under No. 7 can lift 40 000 and 20 000 kg. respectively. In the former case the stronger arm is used, and in the latter the smaller one. These arms are on opposite sides of the crane (see Fig. 2).

This crane is worked by 12 men and has four handles. With the smaller arm 10 000 kg. can be lifted directly, and 20 000 kg. by means

of a movable pulley; whereas, with the stronger arm, loads of 27 500 kg. can be lifted by means of a double block and of 40 000 kg. with a treble block. When the load exceeds 20 000 kg., the crane is balanced by a counter weight. Using the turning gear, two men can easily swing the crane with the weight of the load and the balance weight.

The foundation of the crane consists of concrete. The crane is fixed on a heavy iron sole-plate, and is protected by an iron casing. It is guided at the top in a collar-plate firmly anchored in the masonry.

The cranes under No. 8 require two men, in order to lift the maximum load. By means of a special arrangement the wheels can be thrown out of gear and the chain raised rapidly. By putting his weight against the jib one man can swing the crane. Some of them are provided with swinging gear. The projection and lifting height of the steam cranes, as mentioned under No. 13, were increased.

Deeper harbors were required, especially for transatlantic liners, necessitating the construction of cranes having a greater lift.

The beam of vessels was also in proportion to the increase in their size, so that the projecting jib of cranes had to be increased accordingly. For this reason the new steam cranes were provided with a jib of 9.8 m. (32 ft.) instead of 6.8 m. (22½ ft.), and a lift of 13.75 m. (45 ft.) instead of 10.3 m. (34 ft.), as in those first built.

The top of the jib was raised to a height of 8.15 m. (26¾ ft.), this, however, reaching the extreme limit, as with a further increase a raising of the shed roofs would be necessary, to allow a free swinging of the cranes below them, and consequently this would very much interfere with the dry storage of goods by readily exposing them to rain and unfavorable winds.

The lifting speed of the cranes was 1.15 m. (3¾ ft.) per second, and with a speed of 1.2 m. (4 ft.) the safety valves of the hydraulic check cylinders would be brought into action.

The steam crane mentioned under No. 20 was an object of trial, having been previously altered from the original construction. It was a one-cylinder steam crane, retaining the principle of the set of pulleys.

In place of the hydraulic check cylinder, a friction brake was introduced, which was so designed that it worked automatically if the velocity of the lifting chain overreached a certain limit. It was in-

tended to simplify the construction of the crane and to avoid the inconvenience of draining the water, which was necessary during cold weather when the crane was not in use, in order to prevent any damage through frost. No alteration was made in the bogie or in the turning-gear; the advantages gained by this construction were, however, of no great value, and did not balance the disadvantages which arose in the working through the introduction of a second system. The 15 steam-cranes described under No. 25 and No. 26 were required in augmenting the working material. The length of vessels calling at Hamburg was constantly increasing, and the number of hatches increased in like proportion. The earlier ships had only two hatches; whereas, in modern vessels four hatches are very often arranged. In the interest of a speedy despatch it became necessary to work all hatches at once, thus necessitating an increase in crane material.

The ample supply of cranes for the quays which followed materially furthered a quicker despatch of vessels.

But the transfer of the discharged goods from the quay-sheds into the railway cars, which were placed behind the sheds, was found difficult to perform. Owing to the growing activity it became necessary to have suitable lifting gear. The use of the railway track, situated in front of the quay-sheds parallel to the crane track, was to be avoided as much as possible, and for this reason the transport to the back of the sheds and into the cars had to be facilitated. For this purpose wall cranes were arranged (see under No. 29) close to the back of the shed, with a lifting power of 2 500 kg. The cranes arranged in the modern sheds, having no massive walls, were placed on separate foundations and were held at the top of the crane post in strong iron bands connected with the walls. The lifting gear was arranged so as to allow a change in the lifting, having one set for heavy goods and another set for lighter goods up to 750 kg.; the heavier goods to be lifted by two men, and in all the other cases the cranes to be worked by one man only. These cranes are of particular advantage when the cars are to be filled with light goods at great height, such as wool, etc.

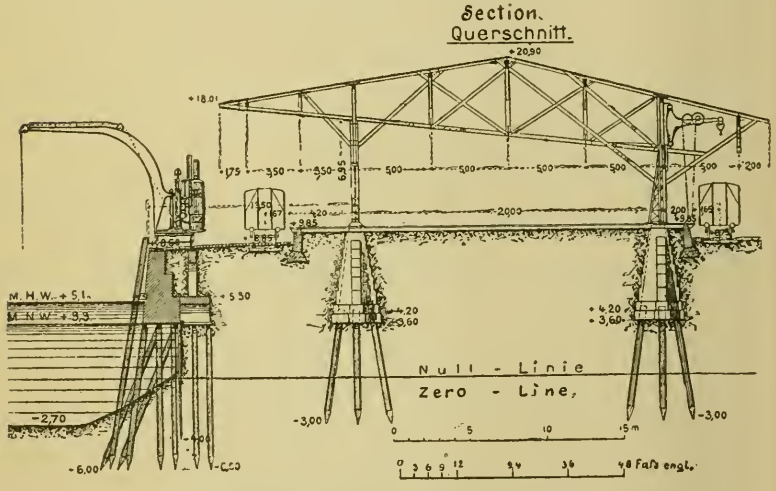
The reduced amount of trade material furnished in the years from 1885 to 1888 corresponded with the building of quays then projected and begun, being the consequence of Hamburg's incorporation in the German custom union. At the end of the year 1888, there were in use 117 steam cranes, which number since that time has been rapidly in-

creased. The first quay lines opened for public use in consequence of the incorporation were the Versmann quay and the America quay. These were furnished with 25 and 12 movable steam cranes, respectively, shown under Nos. 32 and 33 (see Fig. 3).

Welded plates were tried for crane boilers, which were afterwards found to give great satisfaction. Another trial was made with two cranes furnished with wire rope instead of chains.

At this time the lack of means for lifting very heavy loads was felt. Even the existing 40-ton hand crane did not suffice, for instance, in shipping heavy gun-carriages and guns. Therefore a steam crane of

FIG. 3.



Schuppen am Versmannquai.
Shed on the Versmannquai.

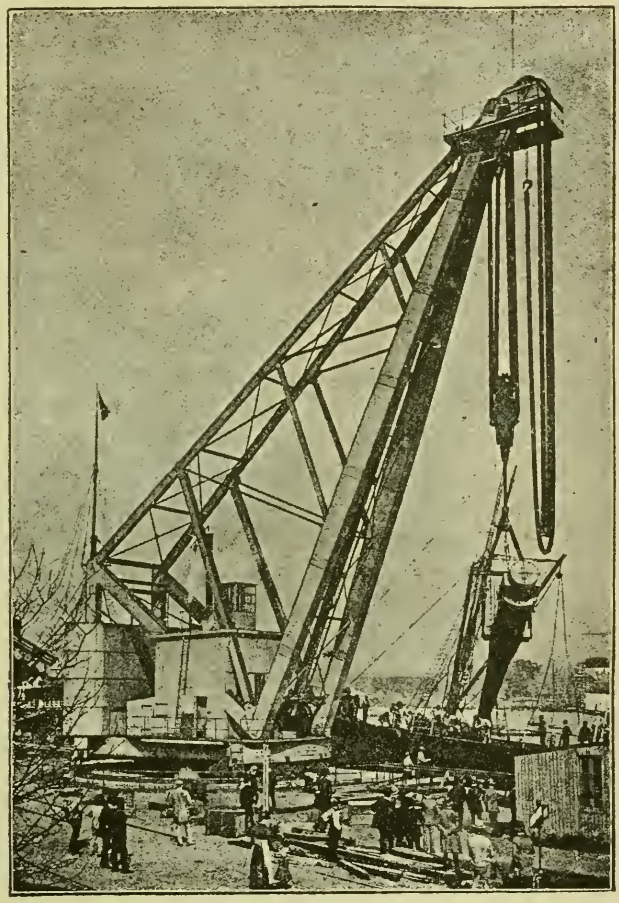
150 tons lifting power was built, in order to meet the growing requirements (see No. 34 of the table, and Fig. 4).

This gigantic crane is the heaviest in use at any European trading port, and is situated at the entrance of the largest harbor of Hamburg, viz., of the "Segelschiffhafen," at a place called Krahnhöft, and is easily reached by ships of any size. The crane is connected with the railway lines, and can be reached by all kinds of vehicles.

The crane swings on a pivot which is fastened to the foundation with eight heavy stays, and moves on 32 steel rollers of 500 mm. (19¾ ins.) diameter and 180 mm. (7 ins.) breadth, traveling on a track of

cast steel; the projection of the crane jib from the center of pivot to the chain being 17.3 m. (56¾ ft.) and 10 m. (33 ft.) outside of the quay wall. The height of the pulley over the top of quay is 31 m.

FIG. 4



150 t-Dampfdrehkrahn am Krahnhöft.

150 tons Steam-Swinging Crane on the Krahnhöft.

(101 ft.). This considerable height was arranged in order to be able to lift or to set up masts of sailing vessels when required.

A ballast tank is arranged, filled with sand, with a total weight of 25 tons; this serves as a balancing weight when lifting.

The boiler and engine are arranged in a separate iron casing of the crane. The boiler has a heating surface of 35 sq. m. (376 sq. ft.) and a working surface of 5 atm. (75 lbs.). The engine is of the double-cylinder type, each cylinder having a diameter of 314 mm. (12 ins.) and a stroke of 550 mm. (22 ins.), and is fitted with a double crank shaft and with Meyer's expansion valve gear. The revolutions are 60 per minute. A flat link chain of steel is used as a lifting chain. The total dead weight of the crane amounts to 230 tons, exclusive of ballast. Weights of 150 tons can be lifted 6 m. (20 ft.) in a quarter of an hour, and one revolution of the crane can be completed in five minutes. The massive crane foundation is based upon 180 piles. The cost of the crane, exclusive of foundation, amounted to about 150 000 marks.

The hand cranes mentioned under No. 35 and No. 36 differ from the former in several respects. First of all, the jib is curved in shape, similar to those of the steam cranes; then, the brake was entirely altered. In place of the iron band-brake there was arranged a centrifugal brake. The apparatus essentially consists of the arrangement of a flying weight (see Plate III), which acts by friction discs, and these act more powerfully the greater the velocity of the weight becomes. By this means a complete and secure regulation of the velocity is accomplished.

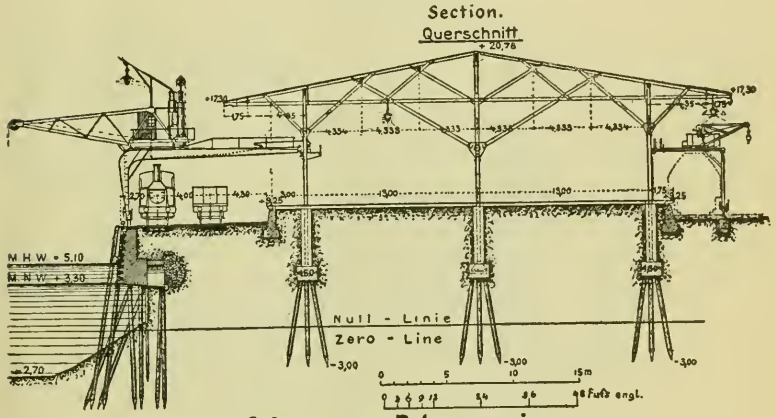
The load remains in uniform motion even if careless handling should cause a sudden accelerated lowering. The driving of the winch is by means of a safety winch handle, *c*, which is always turned in the same direction, being worked either with single or double gearing.

By this means the handling is considerably simplified. Hurling of the winch handle is prevented, also the disengaging of pawls and the shifting of brake levers. By turning the handle the load is lifted; by withdrawing or pressing, it is lowered; and by loosening, it is stopped. The crane-man has only to work the winch handle, and can give his full attention to the motion of the load. In place of lifting chains, wire rope is introduced for these cranes.

For the transport of goods from the quay sheds to the railway car at the back of the sheds, there were arranged for the Versmann quay six wall cranes, shown under No. 37. Contrary to the former wall cranes as furnished to quays, they were built movable. The carriage consists of two separate parts which are arranged one above the other, the

crane-post being fastened between them (see Plate IV). Every part of the carriage is placed on rails, being moved by means of rollers which are conducted along the wall of the shed and securely connected with them. The jib has a section of box form, and encloses the crane post like a mantle. Upon this is arranged the lifting winch, fitted with a safety brake, the latter being of similar construction to that of the hand cranes already described. The winch is worked by a hand wheel and endless chain, and the gear which accomplishes the swinging of the cranes is driven in the same manner. For the lifting of loads a wire rope is fitted. The carriage is moved by means of an endless

FIG. 5.



Schuppen am Petersenquai.

Shed on the Petersenquai.

chain. The motion is received from a chain sheave which is turned by means of a crank shaft.

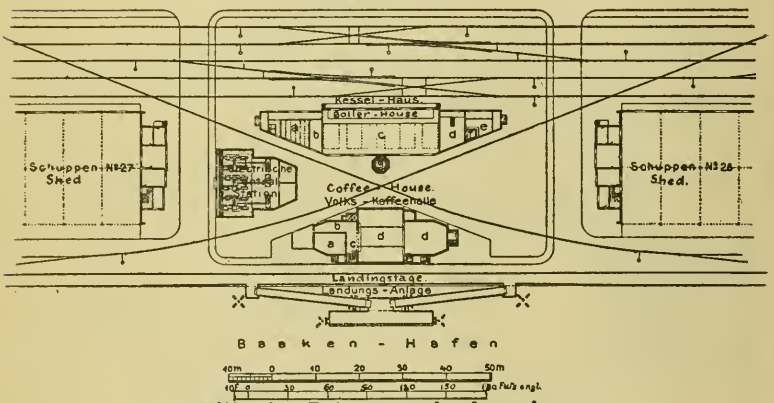
The cranes supplied to the Petersen and Asia quays (Nos. 38 and 39) differ in two respects from the previously described steam cranes.

First of all, the crane carriages received quite a different shape. They were formed in an angle, of which the vertical leg rested upon the quay wall; whereas, the other extends 10 to 11 m. (33 to 36 ft.) horizontally over the top of the road at a height of about 5 m. (16 ft.), being supported at the wall end of quay shed (see Fig. 5).

Here, as well as on the quay wall, rails are arranged on which the crane can travel, having two wheels on each side. Upon the horizontal arm is placed the crane. The main dimensions of the carriage and the crane itself are as follows :

1. Difference in the height of the two rail-levels of the crane track, 5.3 m. (17 ft. 5 ins.).
2. Projection from center of crane to center of lifting rope, 11 m. (36 ft.).
3. Greatest free passage below the crane, 4.95 m. (16 ft. 3 ins.).
4. Projection from edge of quay wall to center of rope, 8.25 m. (27 ft.).
5. Lifting height of the crane for the Petersen quay, 13.75 m. (45 ft.)
6. " " " Asia quay, 18 m. (59 ft.).

FIG. 6.
Plan of situation.
Lageplan.



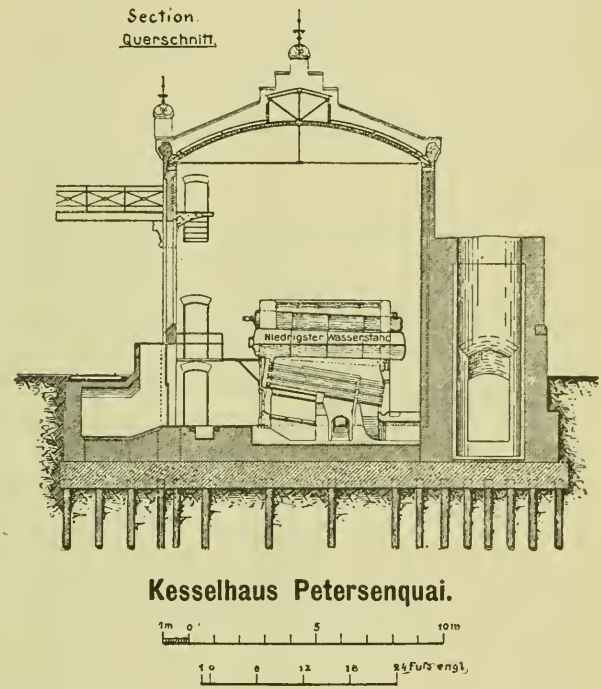
Mitte des Petersenquai. Lageplan.
Centre of the Petersenquai. Plan of situation.

The cranes are not supplied with separate boilers; but the 32 cranes of the Petersen quay and the 27 cranes of the Asia quay are combined respectively with a central boiler station for each, which supply the steam for the cranes.

Both constructions are of the same type and differ only in details. For this reason the following gives the description of the construction at the Petersen quay only. This extends over a length of 1 250 m. (4 100 ft.), and has four sheds, each of which is 260 m. (850 ft.) long. In the center of the quay, between the two middle sheds and parallel to their longitudinal axis, the boiler-house is erected. From this, two branches of steam pipes are laid, each being 625 m. (2 050 ft.) long, to which the movable steam cranes may be connected at any convenient distance of 9 m. (29½ ft.) apart (see Fig 6).

The boiler-house contains six boilers, each having a heating surface of 135 sq. m. (1 450 sq. ft.) (see Fig. 7.) Three of them supply the steam for the 32 cranes and the two 100-H. P. engines which are used for driving the dynamos at the electric station of the Petersen quay. Two of the boilers are for the supply of steam for the engines of the neighboring

FIG. 7.



Boilerhouse.
Petersenquai.

Kirchenpauer quay, to drive the electric cranes, and one is in reserve. As the boiler-room for the six boilers allowed a depth of 9.6 m. (13 ft. 6 ins.) only, it was necessary to introduce water-tube boilers, these being shorter than those with large water space. It was found necessary to introduce a boiler pressure of 13 atm. (195 lbs. per square inch),

in order to maintain a pressure of 10 atm. in the cylinder of the crane which is connected nearest the end of the pipe, at a distance of 625 m. (2 050 ft.).

The general arrangement of the boilers consists of three upper boilers with only one water chamber. The generated steam of all boilers is gathered in a copper receiver of 250 mm. (10 ins.) diameter. The pipes with their branches to both sides of the Petersen quay are connected with this receiver, and can be shut off by special stop valves. Besides these, separate valves of smaller size are connected with the receiver, in order to supply the steam to the auxiliary engines, such as those used for the electric light station, the restarting-injector, and the steam feed pump. All these valves are arranged in the boiler-house.

The steam pipes are conducted over-head on bridges across the railway lines and the road into the quay sheds. The pipes are placed on a gangboard which, in connection with the rail bearers of the crane wheels, is mounted on the columns of the sheds. The steam pipes throughout are of patent welded iron, their internal diameter being at the entrance 100 mm. (4 ins.) and at the end 60 mm. ($2\frac{3}{8}$ ins.). Special care is taken in the connection of the flanges, in order to avoid any leakages whatever.

For this reason the pipes received strong welded iron collars at the ends, these being kept in place by loose iron flanges and bolts, with check-rings between them. The so-called lens-packing pieces are used for check-rings; they are of bronze, and are manufactured in the form of lenses, which are carefully ground into the conical turned seats. Notwithstanding the periodical steam consumption of the cranes, causing violent shocks in the pipe arrangement, these flange connections have given entire satisfaction.

At distances of about 70 m. (230 ft.), there are arranged special water separators, made of wrought iron, and there are two expansion pieces fitted to each side of each separator. Between the separators, the pipes are fitted to small movable carriages, in order to secure a free motion of the pipes in every direction. A steam gauge is fitted to each separator, to indicate the steam pressure at that point. The valves connecting the steam-cranes are arranged 9 m. ($29\frac{1}{2}$ ft.) apart.

In order to prevent the condensation of steam as much as possible,

the pipes, having been heated, were covered with a coat of asbestos 10 mm. ($\frac{3}{8}$ in.) thick. This coat was covered again with asbestos rings of 15 mm. ($\frac{5}{8}$ in.) thickness, 20 mm. ($\frac{3}{4}$ in.) width, placed 50 mm. apart, on this again being laid a close mantle of cork of 40 mm. thickness, served with wire. The spaces formed between the asbestos coat and the cork mantle were filled with a mixture of cork and gypsum. A wrapping of hydrated sulphate of lime was laid round the whole, and this again was mounted with non-conducting material prepared with black japan, and when finished was coated with black japan. This jacket was stopped near the flanges by cast-iron rings. The space between these rings was filled with loose pieces of cork and closed with a divided case of zinc, 50 mm. (2 ins.) thick, filled with cork-stone.

The moving of the frame-formed crane carriage is accomplished by means of hand-gear, so constructed that the two wheels, one being arranged at the water and the other at the land side, can be turned together, or after withdrawing a clutch the wheel on the water side can be worked alone, this being necessary when the cranes are moved through the curved part of the track. The crane carriage is mounted with a turned cast-iron race-ring, on which the upper crane-frame is placed, by four rollers. The frame is guided by means of a steel pin, fitted to the boss of the cast-iron race. A hole is bored in the center of the steel pin, to allow the steam pipe to pass through it (see Fig. 8). One part of the steam crane pipe is fastened to the crane carriage; to this is connected a jointed pipe, fitted at the one end to the above-mentioned main steam pipe and at the other side a small piece is fitted to the steam cylinder, which follows the turning motion of the crane. The stuffing box accomplishing this motion is placed in the axis of the crane.

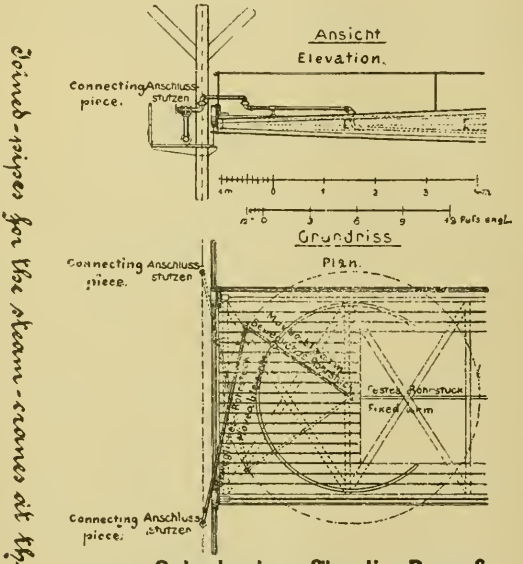
In order to receive an exact result of the efficiency of this central system of crane construction relative to the boiler and pipe arrangement, a series of trials was made in the following manner:

Sixteen of the cranes were tried continuously with the maximum load of 2 500 kg. each, for a period of 9 hours, being supplied with steam from one of the two steam pipes of 625 m. (2 050 ft.) length. Each crane raised the load from the floor of the shed, swung it over the water, deposited it at the water level, raised and brought it back again in the same way. The total lifting height completed during one

turn-play of the crane was about 12 m. (39 ft.), and the total distance swung was about 60 m. (195 ft.). Each crane made on the average 26½ double strokes per hour, but the efficiency of some of them was increased to 40 strokes per hour.

In both boilers kept under steam during this work, 1 kg. of coal evaporated 8.6 kg. of steam of 736 calorics (normal); the utilization of fuel was 70%; the evaporation for 1 sq. m. (10½ sq. ft.) heating surface per hour was 14.6 kg. and the cost of 1 000 kg. of normal steam was 2.45 marks.

FIG. 8.



Jointed-pipes for the steam-cranes at the Petersenquai.

Gelenkrohre für die Dampfkrähne am Petersenquai.

The steam piping condensed 1.55 kg. of water per square meter (10½ sq. ft.) inside surface of the pipes; the non-conducting covering of these diminished the radiation of heat by 38 per cent. There was no loss of pressure in the pipes when the cranes were not working; when hoisting with 10 cranes simultaneously, for one moment the greatest loss of pressure was 1.6 atm. (24 lbs. per square inch). The mean velocity of the steam in the pipes was 18 m. (59 ft.) per second; when 10 cranes worked simultaneously, the velocity of the steam rose to 30 m. (99 ft.) per second.

The cost of moving one ton of cargo by means of a crane connected with this central steam crane station of the Petersen quay was 0.293 marks in the year 1891. On the Versmann quay, however, where each crane has its own fire, the cost was 0.349 marks. Therefore, one ton of goods is discharged 5.6 pfenniges cheaper on the Petersen quay than on those quays without central steam crane stations. Thus, in spite of the high cost of building the cranes, they work more economically.

The building of the central steam crane station on the Petersen quay showed the great advantage which a central distributing station can have in the working cost. On the other hand, the steam pipes showed that there was still a considerable loss of pressure through condensation in them, besides the difficulties of their construction, which had to be surmounted on account of the high pressure and the high first cost. Then, again, the heavy shocks which were produced in the pipes by the periodical taking away of steam by the cranes made careful working a necessity.

These circumstances caused the use of the electric transmission of power to be considered, especially as cables are not only much cheaper than pipes, but also because the loss of power can be reduced at will by altering the section of the cables. Then, again, cables can be laid above or under the ground in a very simple manner, without pipes and without the necessary bridges across streets and railways which pipes require.

A further advantage was expected in respect to the economical use of power when electro-motors were introduced for the cranes, principally because an electro-motor automatically takes a quantity of electricity sufficient for lifting or lowering the load.

As up to the year 1891 nothing was known of electric cranes suitable for the conditions of the Hamburg quays, it was decided for the present to build two trial electric cranes (No. 52 of the table) before providing the whole length of a quay with such cranes. As there was a central station for electric light on the Petersen quay, the size of which was considered ample for the electric trial cranes, these were placed on the Petersen quay.

Both cranes resemble the steam cranes on the Petersen quay in their principal dimensions and in the weight of the load to be lifted. The construction of the carriage differs very little from that of the steam cranes, and they are moved like them along the quays by means of

hand gear. However, the lifting and swinging gear, and also the brake, have been considerably altered on account of the rotary motion common to electric motors.

One crane, construction A, has a winding engine, consisting principally of a treble-threaded worm working in an oil trough, which turns a worm wheel, on whose steel shaft the cast-iron rope drum is fastened. Worm wheel and rope drum are fitted between two strong cast-iron stands, which are fastened to the cast-iron platform and placed as far back as possible in order to act as a counterweight to the jib and the load of 2 500 kg. hanging to it. The tension stays of the jib are bolted to the top of the stands, and the foot of the compression stay to the platform. The pulley for the steel wire rope is fixed at the top of the jib, and is made as large as possible. One end of the wire rope carries the hook, which is provided with a buffer, and the other end is either wound or unwound on the drum, which has turned grooves to take up the rope. The shaft of the worm is coupled direct to a joined 40 H. P. electro-motor, making 500 revolutions per minute by means of an elastic steel brush coupling. A wedge brake is fitted to the worm shaft and can be worked by the controlling lever ; the worm shaft, and therefore, the drum and load, can be stopped instantaneously, and the elastic brush coupling at the same time allows the armature to revolve for a short time, till its energy is consumed. By this means the armature and the different parts of the winding engine are not overworked and damaged. The high threaded worm gearing, whose efficiency after the building of the crane was ascertained to be 70%, has the advantage that the worm turns back of its own accord and turns the electro-motor when the load on the hook descends. However, as the shunt electro-motor, according to its peculiar nature, always remains at its normal speed, it acts in this case, not only the part of a speed brake, but also utilizes the excess of turning power, producing a current and sending it along the wire. Besides this speed brake, which acts by the nature of the electro-motor, there is another special, electric safety brake for cases when the current is interrupted.

It consists of a specially strong brake wheel with a wedge brake, and is fastened to the shaft of the drum next to the worm wheel. The unequally armed brake lever carries on its short arm the brake blocks and on its other arm a weight, which presses the brake into the groove

of the brake wheel. Besides this, there is an electro-magnet under the brake lever which is surrounded by the wire in such a way that it attracts the lever and loosens the brake when the circuit is closed. However, should the current be interrupted in any way, the electro-magnet loosens the brake lever, the brake blocks fall into the groove and hold the drum and with it the load.

The platform of the crane is made hollow and rests upon three wheels, which run on a turned iron ring on the carriage. The starting platform is pivoted on a hollow steel pin, which also contains the wires for the electric current. The swinging gear of the crane is placed inside the platform. It consists of a separate electro-motor of 6 H. P. with a series of circuit closers, which is connected with a worm shaft by a steel brush coupling, and it turns the front or driving wheel of the platform. The motor is provided with a series of circuit closers, in order to drive it at any speed besides the normal speed, which it attains by the velocity of the load of 2 m. There are two levers resembling the starting levers on locomotive engines for the lifting and swinging gear.

The regulating lever of the winding engine acts, not only upon the brake on the worm shaft, but also upon the resistance of starting, and upon an electric switch, which starts, reverses and stops the motor. Besides this, the simple mechanism of a brake wheel turns the starting lever automatically to its middle position and thus stops the whole winding engine, when the attendant releases the slipper handle or the lever handle.

The starting lever of the swinging gear controls the reversing of a Sprague switch, which gives the motor its several directions of rotations. This lever is also connected with a similar brake gear to that of the winding engine. Both levers are worked by one man, and the hoisting and swinging can be done simultaneously by them.

The current is conducted to the crane by means of a copper contact race, fastened to the pillars of the quay sheds. In this way it is possible to put the crane in any position on the quay without having recourse to peg switches, etc., for the cables.

The other crane, construction B (see Plate V), has quite a different winding engine and also different swinging gear to the one described above. Instead of worm gear it is fitted with spur wheels, and the swinging gear consists of a combination of worm and wheel gearing.

The winding and swinging gear are both placed on the cast-iron sole-plate, which carries the wrought-iron jib.

The electro-motor, being the heaviest part of the winding engine, was placed quite at the back, to balance the crane. The motor makes 500 revolutions per minute, and transmits its movements to the drum by means of a pair of geared spur wheels with inclined teeth.

The great circumferential velocity of the drum necessitated the use of rope driving with a movable pulley, which has the advantage of a thin wire rope and a short rope drum and brake. A safety brake is fitted with a special brake lever which stops the load at once when the attendant carelessly lets go the lever, or if an interruption of the circuit should suddenly take place. It consists principally of a brake sheave fitted to the drum shaft and jammed by a balanced weight lying between two brake blocks. When the crane is not working, the rope barrel is automatically thrown into gear with the jammed brake wheel; whereas, when the load is lifted this coupling disengages itself.

The starting platform of the crane rests with four wheels on a turned iron race ring, fastened to the carriage. The swinging gear is on the starting platform between the large spur wheel and the brake wheel and below the rope barrel. On the shaft of the electro-motor with its series of circuit closers is a worm which drives a horizontal worm wheel. A vertical shaft, whose bottom end passes through the platform, carries at its end a spur wheel, which is geared to the inside teeth of the ring on the carriage. The platform has, like the crane described above, a steel pivot in its center, through which the wires of the motor pass.

For working the crane there are three levers, so as to have the same arrangement as on the steam cranes. One is for lifting, one for the brake lever (used at the same time for lowering the load) and one for swinging. The lifting and brake levers are to the right and the swinging lever to the left of the attendant.

By moving the lifting lever forward the load is raised, and by moving it backward the load is held fast. The same movements have to be made with the brake lever for lowering and holding the load. In order to let the electro-motor work as a generator of electricity and send back a current along the wire when lowering the load, the lifting lever has to be moved forward after the brake lever. In order to hold the load again both levers are turned back again. In all these movements the

starting resistance alone is controlled by the lifting lever and the switch by the brake lever.

By means of this swinging lever ensues, at the same time, the connection of the electric motor, and of the resistance as well as the change of connection.

The lever is in its middle position when the crane is not working. By moving it forward or backward the motor is reversed by changing the poles of the magnets, and the several turning directions of the armature are accomplished in this way. Contrary to the crane described first, the connection with the main cable is made by peg switches, and the free ends of both crane cables are joined to the main cable in special closed boxes at a distance of 9 m. from one another.

The crane, construction *A*, has been working for one and a half years, and the crane, construction *B*, for one year. They have proved successful during this time, especially in periods of severe frost, when they showed the advantage of their dry working over the steam cranes. Before the cranes were used they underwent a large number of trials, the results of which are tabulated in the diagrams. With a maximum load of 2 500 kg. every crane worked three hours continuously and completed 45 double strokes. There was no heating of the machinery in any of the cranes.

It follows from the diagrams of current consumption that the current produced by the load, particularly in loading, viz., in moving the goods from the quay sheds down into the hold of the ships, was very strong and that there was a saving of current of over 50% (see Plate VI). When it is considered that two-thirds of the goods discharged from ships at the quays of Hamburg are moved again on lighters, the importance of this return current for cranes, worked from a large central station, will be clearly seen. As a rule, a certain number of cranes are loading, and on the other hand the others are discharging at the same time. In this case the return current produced by the loading cranes flows directly to those discharging and discharges the dynamos in the central station. Accumulators are placed on the Petersen quay in connection with the electric station. They are charged when the two electric cranes are working. The return current produced by the cranes is used in the smaller works to relieve the dynamos charging the accumulators.

It will be clear from the foregoing description that the dynamos can

be made smaller, and that there is a great saving in building and working expenses with these cranes compared to the steam cranes of the same efficiency.

The semi-arch hand cranes shown under Nos. 45 and 46 are fitted with disengaging gear, in order to accomplish a quicker lifting of the vacant hook. These cranes work with wire rope. There are arranged corrugated sheet-iron cabins for the attendant working the semi-arch cranes, to protect him from wind and weather. These shelters are not required for those cranes which are arranged at lower places of the earlier quay constructions. The semi-arch cranes No. 47 were arranged in place of the wall cranes previously in use. They throw a bridge across the railway track.

All wire ropes are worked with a factor of safety not less than six. They are fitted with lift guards, in order to prevent jamming with the top pulley of the jib.

The steam cranes shown at No. 53 have boilers made of mild Siemens-Martin steel.

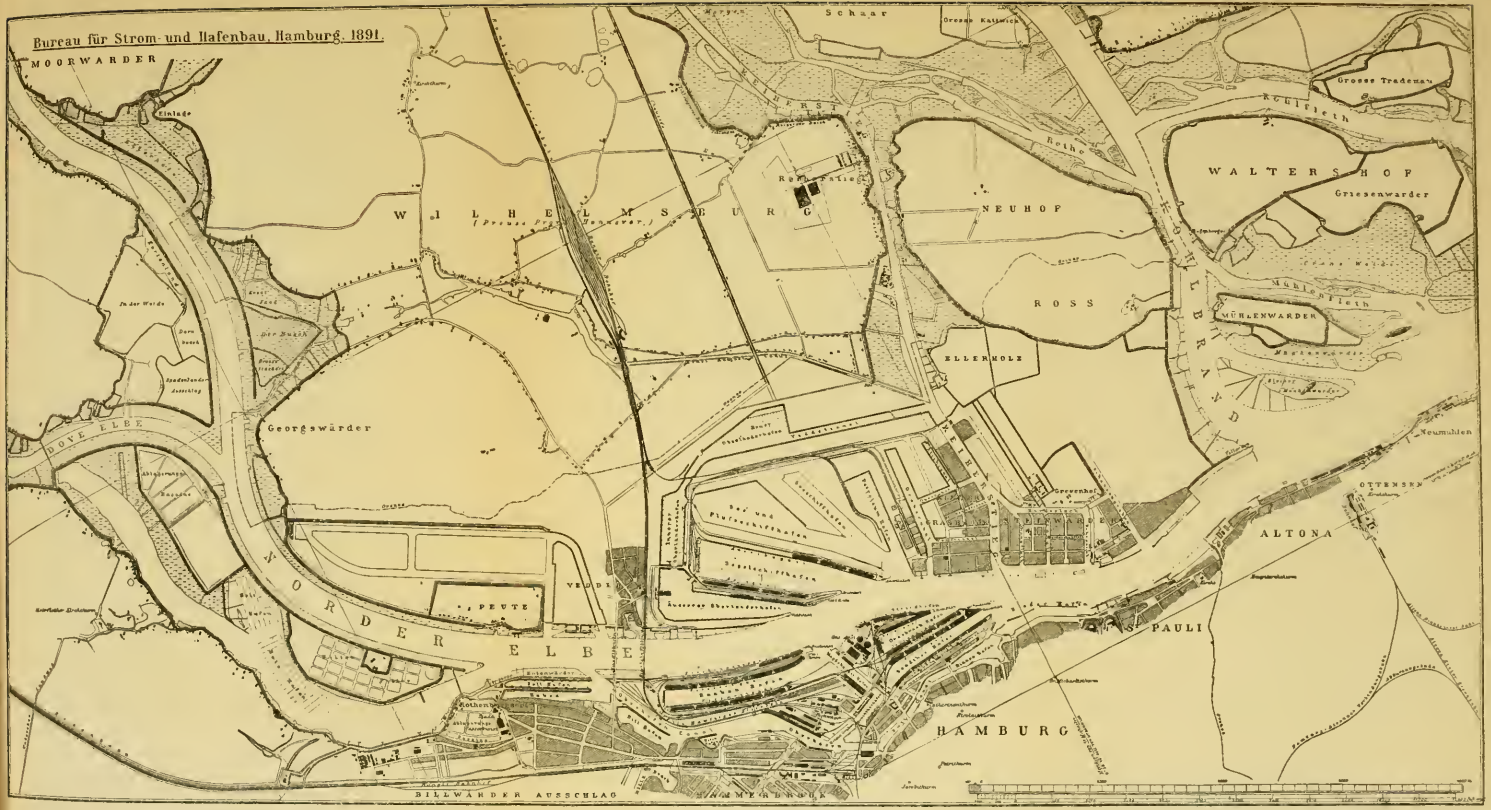
The semi-arch steam cranes shown at No. 54 were fitted with automatic weigh bridge and with regulating gear. These self-acting apparatus have proved very successful, having been previously supplied to cranes. The weight of a semi-arch crane amounts to about 35 tons.

The steam crane No. 55 was furnished both with an iron band brake and a patent centrifugal brake. The latter comes into action only when there is neglect to duly use the former. The crane works by means of a rope, and is moved by the driving gear of the engine.

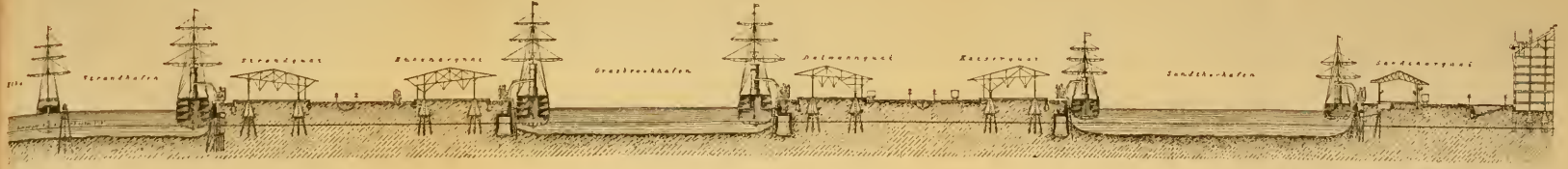
The 50-ton steam crane No. 56 is a turning crane fitted with balanced weight for the load to be lifted.

The crane rests on 16 rollers, which are connected to each other by means of side levers, so that eight rollers always receive approximately the same pressure, the maximum load for each roller being 24 tons. The crane is fastened to the foundation with grappling irons and turns round a pivot and swings through a whole circle in both directions. Loads of 25 tons can be lifted at a speed of 1.8 mm. (6 ft.) per minute, and loads up to 50 tons can be lifted at half that speed.

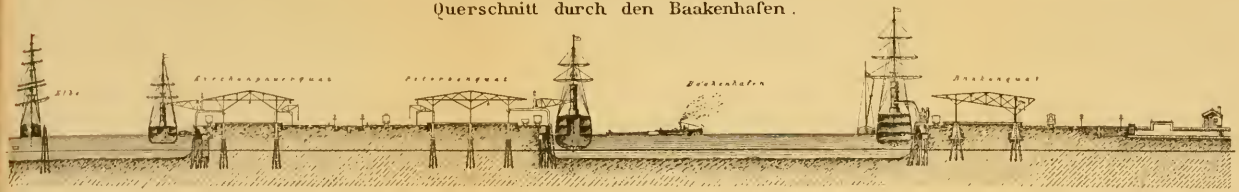
Boiler, engine and lifting gear are directly connected to the crane. The boiler has a heating surface of 10 sq. m. (108 sq. ft.), the engine being a vertical twin-engine with double-crank shaft. The diameter of cylinder is 210 mm. ($8\frac{1}{4}$ ins.) and the stroke 314 mm. ($12\frac{3}{8}$ ins.).


Bureau für Strom- und Hafenbau, Hamburg. 1891.
MOORWARDER
Einlage
Schaar
Grosse Kattwick
Grosse Tradenau
Rohlfleth
Rothe
WALTERSHOF
Griesenwarder
WILHELMSBURG
NEUHOF
ROSS
Mühlenfleth
MÜHLENWARDER
ELLERHOLZ
Georgswärder
DOVE ELBE
NORDER ELBE
PEUTE
VEDDE
Neumühlen
OTTENSEN
ALTONA
PAULI
HAMBURG
BILLWÄRDER AUSSCHLAG

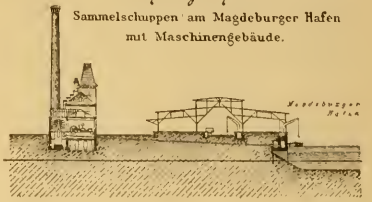
Section through the Quays at the „grossen Grasbrook."
Querschnitt durch die Quaianlagen auf dem grossen Grasbrook.



Section through the „Baakenhafen."
Querschnitt durch den Baakenhafen.



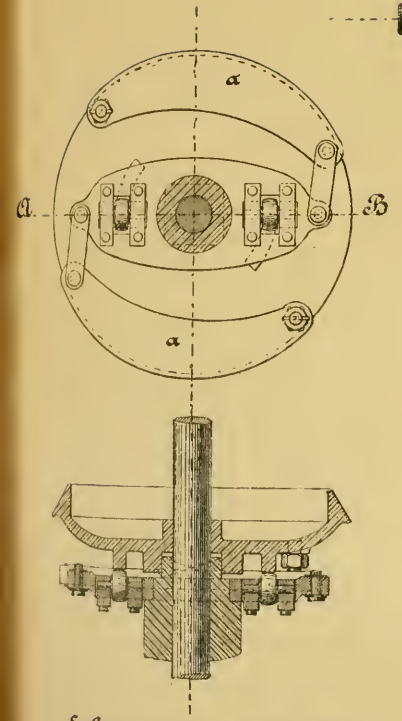
Central shed at the Magdeburgerhafen with Enginehouse.
Sammelschuppen am Magdeburger Hafen mit Maschinengebäude.

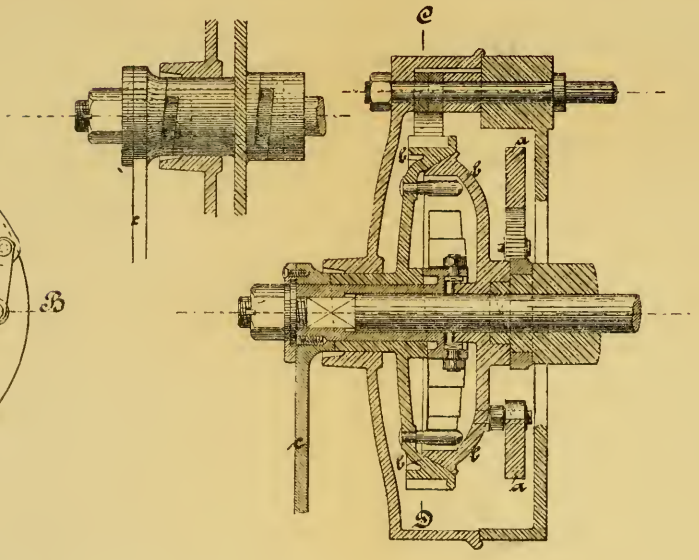


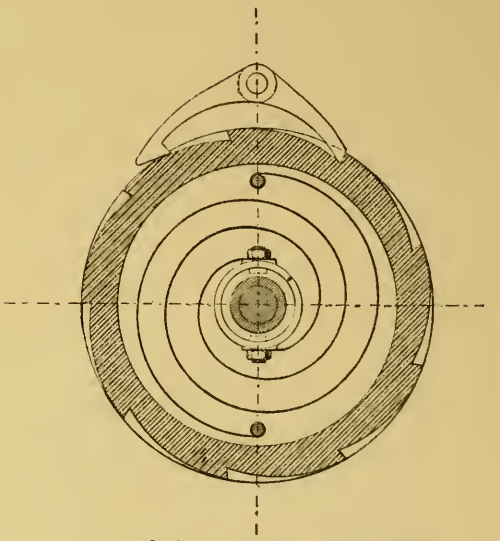
Section through the „Segelschiffhafen."
Querschnitt durch den Segelschiffhafen.

Sicherheitskurbel mit Centrifugalbremse.

Safety crank with Centrifugal-brake.





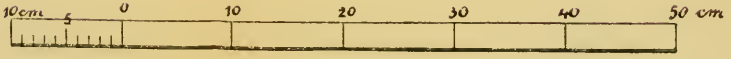


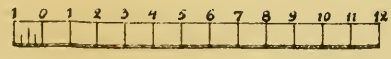
Schnitt nach C – D.

Section C-D.

Schnitt nach A – B.

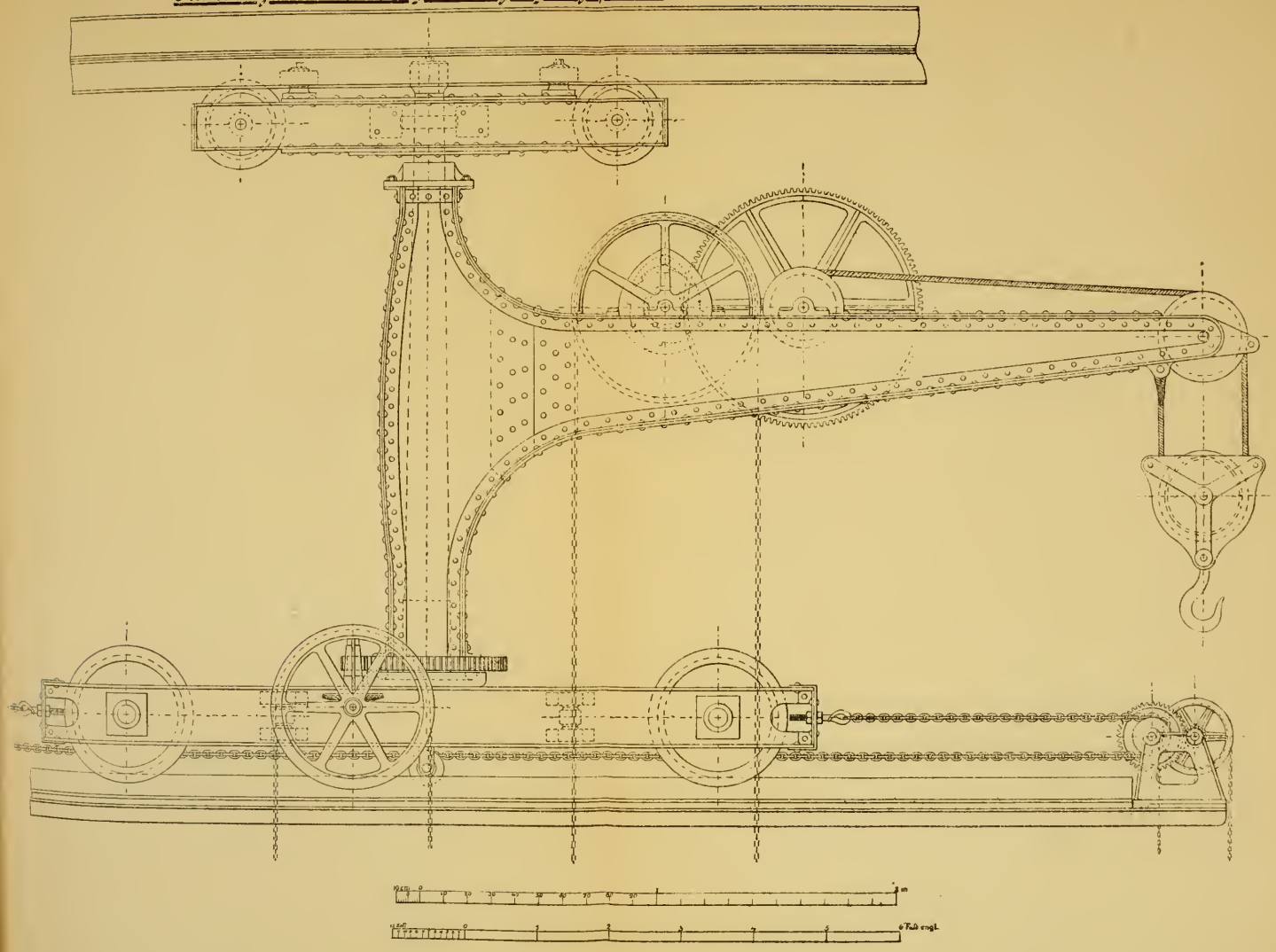
Section A-B.

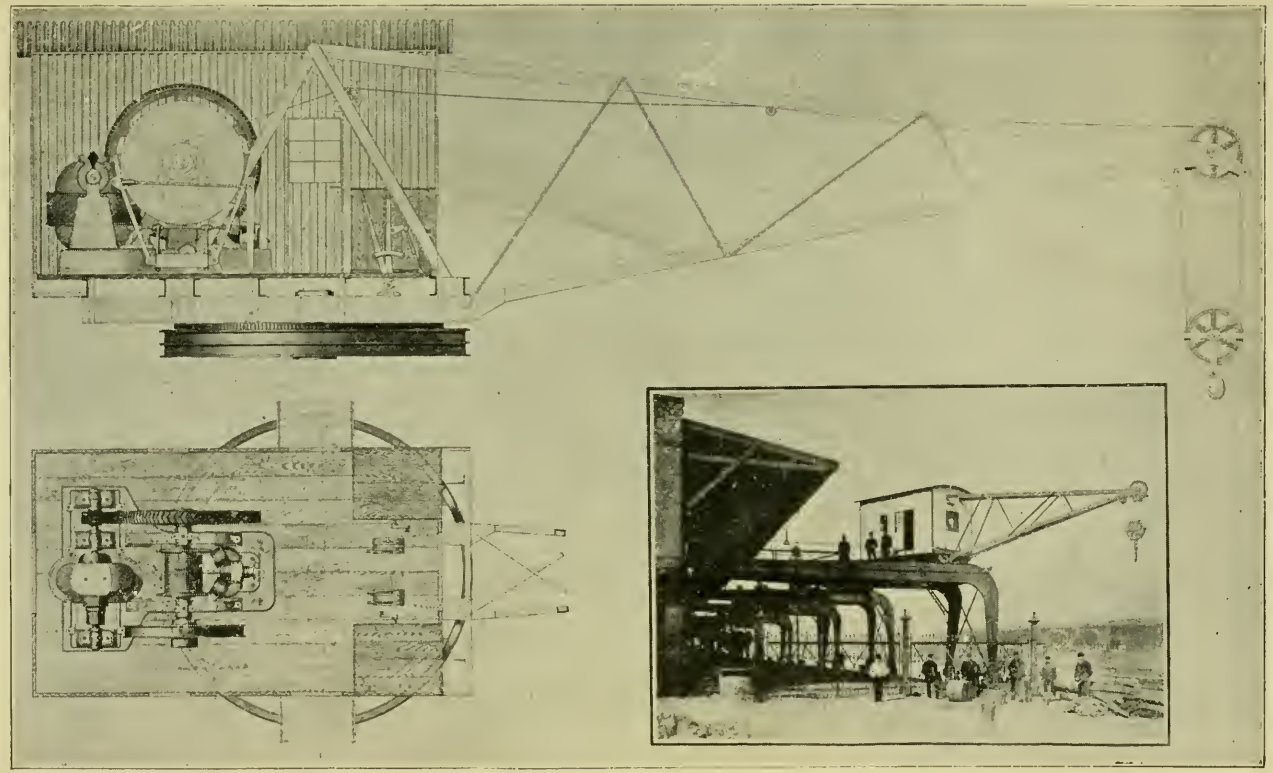




Fahrbarer Wandkrahn für 2500 kg Last.

Travelling Wall-crane of 2500 kg Lifting-power.





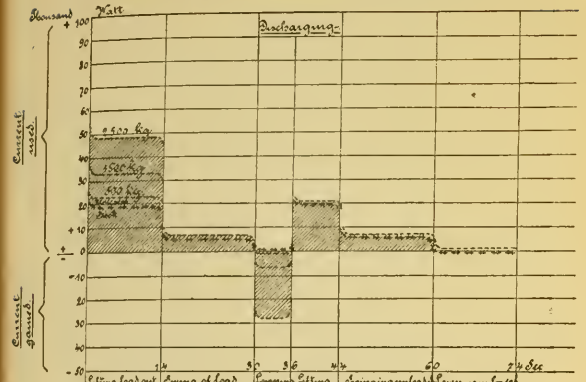
Electric Crane. (Construction B.)

VOL. XXX, No. 545.
TRANS. AM. SOC. CIV. ENGRS.
NEHLS ON DEVELOPMENT OF QUAY CRANES AT HAMBURG.
PLATE V.

Electric Swinging-Crane.

of 2500 kg Lifting-Power on the Petersenquai.

Diagram of a complete double Stroke.



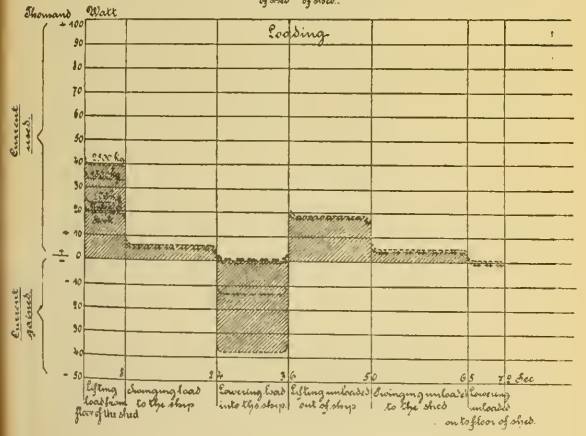
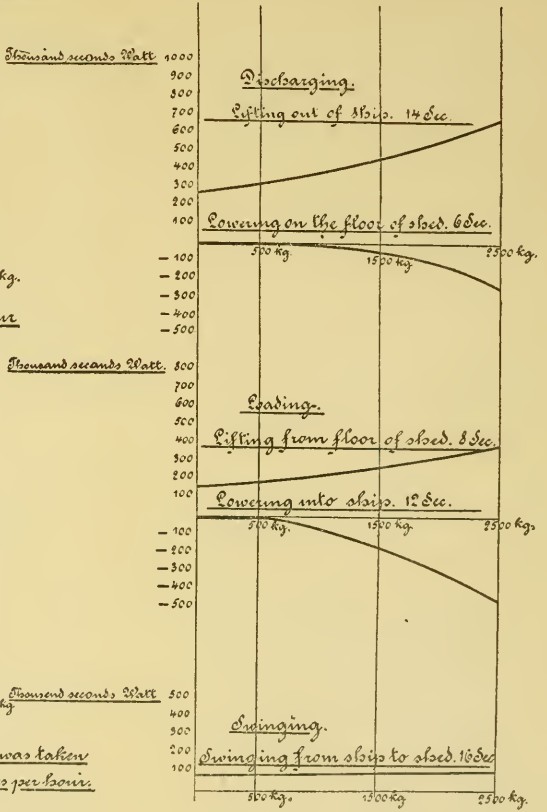
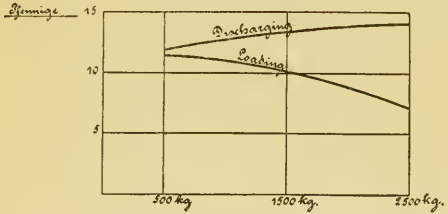


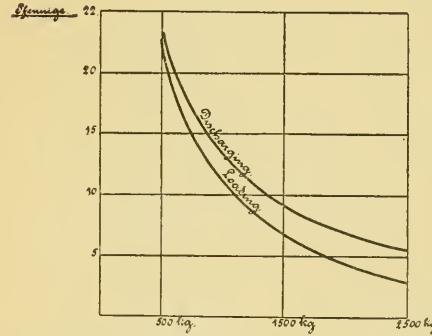
Diagram of the single Movements of a double Stroke.



Diagrams of the Cost of Coal per hour for the Load.



Diagrams of the Cost of Coal per hour for 1 ton.



Remarks: When making the diagrams of the cost of coal it was taken for granted that one crane makes 24 double strokes per hour.

The engines work with Fink's expansion link motion. Weston's patent brake is fitted, in order to prevent untimely lowering of the loads; this does not offer any resistance by lifting, but will be put into action by lowering. The loads are lifted with wire rope. The cost of the crane amounted to 81 000 marks.

These are all of the crane constructions which have been completed up to the year 1892.

It will be seen from this communication that all alterations in the construction have been avoided as much as possible, both with the steam or hand cranes. Only absolutely definite improvements were taken into consideration which were found necessary during working. The more variations that are made in the construction, the more difficult it will be found to carry on the repairs and to furnish the necessary compensations.

In future the furnishing for the new harbors now in course of construction will be accomplished in the same manner, partly consisting of steam cranes constructed on the same principle with a set of pulleys and hydraulic check cylinder, and partly of electric cranes.

There exists no inclination for hydraulic cranes, as they are endangered during the winter time through frost. The few cranes of this type at present in use are situated almost exclusively at protected places. The climate of Hamburg does not seem to be favorable to this kind of motor, although it is not to be overlooked that these cranes possess advantages in having simple gearing, and consequently there is very little cost in repairing them.

For the reason above given, electricity and steam will be the future motors for the crane constructions of Hamburg.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

646.

(Vol. XXX.—November, 1893.)

METHOD USED TO SECURE THE STABILITY OF A QUAY WALL AT THE PORT OF ALTONA, ON THE ELBE, IN GERMANY, WHICH HAD SHIFTED ITS POSITION AFTER COMPLETION.

BY BERTHOLD STAHL, Stadtbaurath at Altona, Germany.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

Altona is situated on the right bank of the River Elbe, just below Hamburg, and closely connected with that town. It has 150 000 inhabitants, who earn their living by the commerce and industry of the town.

The old-fashioned quays were not adequate to the needs of commerce and the rapidly developing fishing trade, which for the last few years has been in a flourishing condition. The depth of the harbor was not sufficient for steamers of great draft, so that the latter were

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

obliged to discharge their cargoes into lighters at some distance from the harbor. For two reasons it was out of question to think of deepening the harbor so that ocean-going vessels could be moored alongside the quays and loaded or discharged there. First, the foundations of the old quays were not deep enough to give the necessary depth of water at their sides; and second, the warehouses on the quays reached to the water's edge, so that it would have been impossible to make use of the deepened quays.

In order to get over this difficulty, the town of Altona spent a large sum of money, and bought ground to the extent of 550 m. (1 804 ft. 6½ in.), demolished the private warehouses, erected new quays, with deep foundations, dredged the harbor to a depth of 4.6 m. (15 ft. 1 in.) below the low-water level, and built a railway and sheds on the quays, so that Altona has now a modern harbor, which meets all requirements of the present time.

The discharging and loading at the quays is done by means of traveling cranes, which lift the goods out of the vessels and deposit them in the sheds or railway trucks (and *vice versa*) without any loss of time. The ground on which the quays were erected is very unfavorable, both as regards the kind and the formation of the soil.

The section (Fig. 1) shows the ground, which rises from the Elbe and attains the height of 32 m. (105 ft.) at a distance of about 500 m. (1 640 ft. 6 in.) along the River Elbe. The town is built partly on the slope and partly on the top of this rising ground.

The ground for the quays had to be artificially raised for a width of about 100 m. (328 ft.) (shown at *A* in Fig. 1), and this was protected from the river by masonry above the high water-mark (shown at *B* in the same figure).

The top layers of the ground consist of very fine sand with numerous water-streaks. Below this sand is a firm layer of clay, which rises to the hills at a smaller angle than the surface. The clay is firm when not soaked by water. It is dissolved, however, by the water, and is then a smooth, soapy body, which allows the top layers to slide down.

As long as the water from the hills can percolate freely and without hindrance through the sand, there can be no fear of a catastrophe. As soon, however, as the free outlet of the water is stopped, the body of sand becomes papescent, and the clay becomes lubrical (soapy) on its surface, and landslips occur.

When the wall was built, these facts must have been ignored, or, at any rate, not considered dangerous. The wall was built and constructed nearly on the same lines as the one in Hamburg, but it must be remembered that the formation of soil in Hamburg was quite different, so that although it suited the conditions of the ground there, it did not prove successful in Altona.

Fig. 2 shows the construction of the quay clearly.

The front part of the wall is solid and backed by pillars placed 9 m. (29 ft. 6 in.) apart, center to center, and secured to it. Between these pillars there are arranged two arches (see view from the bank), the bottom one being loaded by sand at the back, and the top one being used as a foundation for a girder to carry the rails of the steam cranes. The rail *R* (Fig. 2) was dispensed with later on, because the front wheels of the cranes run on the edge of the quay and the back ones on a rail on the first floor of the sheds. The cranes are, therefore, semi-arched, and the railway trucks can run underneath them, alongside the quay wall. This very successful system was first adopted at the port of Bremen on the River Weser. The advantages of these semi-arched cranes will be easily discernable. The wall is built of hard bricks and cement mortar (1 part of cement to 4 of sand) and fitted with all the necessary gear. It is built 5.8 m. (19 ft.) above the ordinary water level.

At the lowest ebb tide the crown of the wall is 6.8 m. (22 ft. 4 in.) above the low-water level.

The foundation consists of a pile framing, made of vertical and oblique piles, the tops of which are 8.9 m. (29 ft. 2 in.) and 9.7 m. (31 ft. 10 in.) below the ordinary water level. A tongued and grooved piling is built behind the wall 7 m. (23 ft.) high (see Fig. 3). These piles were driven with very great precaution. The bearing strength of each pile was calculated from the respective depth it was driven in. On the piles is fixed a firm frame, composed of longitudinal and cross beams mortised and connected to one another. Strong boards are placed on these cross beams, which support the wall. The framing is 45 cm. (1 ft. 6 ins.) above the mean water level, so that it could be carefully built above the water-mark, when the water was low.

With the help of this description and the drawing, the following will be clearly understood.

The whole length of the wall (550 m., or 1 804 ft. 6½ ins.) was com-

pleted in two years without an accident; and after filling up the space behind, warehouses (sheds) were built 8 m. (26 ft. 3 ins.) behind the quay wall. These warehouses are light buildings made of corrugated iron, as will be seen in Fig. 1; and the foundations are made so that the wall is not subjected to any lateral pressure from the buildings.

In order to drain off the water of the hill and the street, a drain was laid in the road behind the sheds, and the water was emptied into the Elbe by means of a pipe laid at right angles to the wall (Fig. 4). The drain was firmly secured to the quay wall.

At the beginning of August, in the year 1890, soon after the completion of the quay, cracks were discovered in the wall about 120 m. (393 ft. 8½ ins.) long, which differed from those hitherto found in quay walls in their position, number and size, so that it was at first feared that the wall had not sufficient stability. At the same time it was noticed that the solid gable walls of the sheds placed at right angles to the quay were cracked and giving way, and also that the pillars supporting the roofs of the sheds, and placed on the water side of the same, were not standing vertically, but leaning backwards (to the land side), they were no longer parallel to the lower pillars of the sheds on the land side.

In consequence of the simultaneous appearance of this double phenomenon, it was concluded that, not only the sheds which, as mentioned above, were built on a separate foundation, but also the quay wall itself, had moved forward. It was assumed that the whole earthwork had moved forward, because the pillars at the front of the sheds inclined considerably towards the back, so that their bottom ends had been shifted towards the water, and because the pillars at the back of the sheds had only slightly lost their vertical position. This was proved conclusively by the cross cracks in the drain pipe, placed at right angles to the quay wall. Naturally, these cracks were larger near the wall and diminished in size towards the sheds (hillside).

Without searching for the cause, it was decided to remove the pressure of the earth on the wall without delay, in order to save time. The backing was taken away, so that the wall was laid bare.

Then the front foundations of the pillars of the sheds were secured to the back ones by strong stays of iron, so that the front and back pillars had to move uniformly (see Fig. 1).

It was very soon noticed that the sheds had stopped moving. The

back pillars were bedded in the firm clay which, as has been shown above, rises from the Elbe up to the hills. By means of these stays the influence of the top shifting layer upon the front of the sheds was removed, as the back foundations completely resisted this sliding. In the mean time there was little fear of the wall falling down, as the back pressure had been taken away, and it was now possible to measure the ground periodically, which, together with extensive examination of the soil, would explain the cause of the shifting.

The examination of the soil was accomplished by means of earth-borers and lining the several holes with iron pipes, in order to observe the fluctuations in the rise and fall of the ground-water. The influence of the water from the hill, and its connection with the rising and falling of the Elbe during ebb and flood, were particularly investigated.

First of all, the measurements which were taken with the view of examining the moving of the quay wall, and which had to be made very carefully and very often, will be described.

The following points were determined:

1. The height of the wall crown by leveling. Very little differences were noticed here.

2. The inclination of the wall. In the design, the inclination of the front surface of the wall was fixed at 1 in 10.

By means of an apparatus made expressly for determining the change of inclination, it was noticed that the wall swung gently like a pendulum round a point of the piles, especially in the direction of the water.

3. The most difficult measuring was the observation of the traveling motion of the wall towards the Elbe, and this was done by means of a theodolite. This difficult measuring was rendered still more so by the wall adopting an irregular concave form, and also by the fact that it was always changing its position and therefore impossible to fix the instrument anywhere on the ground. This difficulty was overcome by placing the instrument between two lightning conductors, which were fastened to some high steam chimneys at a long distance, one of them to the west, the other to the east, of the ground to be examined. In this way an immovable vertical plane had been gained behind the wall which was independent of the instrument fixed on the moving ground, and from this plane the distances of the several points

of observation could be measured at any time. In order to prevent mistakes of the instrument entering into the dimensions, it was always fixed exactly in the same position. Then, again, the precaution was taken of having the same engineer make the measurements.

Iron marks were fixed on the crown of the wall, by which means it was possible to place measuring rods in their exact position. The total number of marks was 38, and they were fixed at a distance of 10 to 20 m. (33 to 66 ft.) from each other. Each mark had its own measuring rod marked with its respective number, and, before measuring, these rods were placed in their proper places. The rods carried an iron point at one end, fixed in a vertical position. This point was brought in the plane of sight, and the rod kept always in the same horizontal plane at right angles to the plane of sight. When the first measurement was made, there was fixed on every rod the distance from the plane of sight to the mark placed on the wall, so that, when measuring again, the difference of the first measurement could be accurately read off as soon as the rod had been adjusted. As the dimensions were comparatively small, mistakes could not occur. This method has proved successful during three years' observation and has given reliable figures of the movements of the quay.

It would lead us too far to give a description of the observations made during several years; indeed, it would be almost impossible to show all the tables and graphic representations of all observations in this paper. It will suffice the practical engineer to learn that the data collected showed the cause of the malady and enabled the proper remedies to be applied.

It turned out that the whole body of earth up to the hill above the layer of clay moved towards the Elbe and pressed with such force against the quay wall that the latter was shifted also in the direction of the harbor. However, as the bottom ends of the piles were bedded in the blue clay, a kind of pendulum-swinging round the ends of the piles, took place at first, and this had to be stopped.

This could be done by very long and strong stays, fixed at the top of the pile-construction and connected with the clay hill at the land side. This will be seen from the section in Fig. 3 of the construction, and also Figs. 4, 5, 6, 7, 8 and 10.

Then it was advisable to keep the backing of the wall as light as possible and to have the ground well drained, in order to equalize the

water level of the river and the ground-water level before and behind the wall as soon as possible, and so avoid all unfavorable loading behind the wall and all slipping on the layer of clay where possible.

Before starting this important work, the opinion of the well-known and highly reputed engineer, Herr Ober-Bau-Director Franzius, of Bremen (known in connection with the correction of the Weser), was asked. He agreed with the proposals, and the work was executed according to his design.

The Stays.—In order to secure the whole moving stretch of land to the stays, two thick beams had to be fastened beneath the surface of the water (Fig. 3) in front of the piles, for the purpose of transmitting the pull of the stays to the piles. This difficult task could only be done at low water, and was therefore very slow. At the same time ditches, about 1.5 m. (4 ft. 11 ins.) broad and 5 m. (16 ft. 5 ins.) deep and parallel to one another, were dug between the boards at right angles to the quay wall, at a distance of about 4.5 m. (14 ft. 9 ins.). The water was pumped out by steam pumps. Then a ditch about 5 m. (16 ft. 5 ins.) broad and about 50 m. (164 ft.) from the wall and parallel to it was dug, into which the other pits opened. At the intersections of these longitudinal and transverse ditches, which reached as far as the firm clay, larger pits were made and filled with strong beams of about 2.5 x 2.5 = 6.25 sq. m. (8 ft. 2½ ins. x 8 ft. 2½ ins. = 67.24 sq. ft.) surface, which were supported by the vertical planished front side of clay.

The iron stays were secured between these two points, viz., the outside of the two longitudinal beams placed before the wall and the wooden shield (Fig. 7 and 4) in the clay hill on the bank. These stays consisted of separate pieces (links) 5 m. (16 ft. 5 ins.) long, alternately round bars 8 cm. (3.15 ins.) thick and two flat iron bars of the same strength joined together by a bolt through an eye at their ends, so as to form a chain (Fig. 11). The front link of this chain was first of all pushed from the water underneath the wall and between the piles of the foundation till the front bolt of the stay came in contact with the heavy iron plate resting on both horizontal beams. In order to tighten this chain and the screw behind the shield, the links of the chain were bedded in a closed wooden box, so that in case the earth should slip while the work was in progress it would still not interfere with the mobility of the chain. After all the links of a stay were joined together, a task which often required 3 or 4 weeks, the screw on the

bank was tightened, and in this manner one point of the wall after another firmly secured to the immovable hill of clay.

In all there were fixed 29 stays at a distance of 4.5 m. (14 ft. 9 ins.), and the time spent on the work was about 18 months, which is accounted for by the fact that it had to be done mostly under a heavy rush of water and only at low water. Another great obstacle was that most of the stays had to be passed underneath the finished warehouses.

The Drainage of the Ground.—Besides the method of securing the wall as described above, the draining of the ground behind it was of the utmost importance. First of all, holes were made in the grooved piling under the wall and also in the wall itself at several places and in different heights. Then the bottoms of the ditches, in which the stays were bedded, were filled with stones so that a splendid drain was formed behind the wall from the slope of the hill to the Elbe by which the water could be run off rapidly and without danger. The observations of the water-level made later on, have proved that the object sought has been attained.

The Strengthening of the Quay Wall.—When the quay wall was exposed by the removal of the backing it was seen that the arches (*a* and *b* in Fig. 2) of the wall between the pillars had been totally destroyed. It was not advisable to renew the arches, because the pressure of the earth had unfavorably strained the wall in a vertical direction. Some of the supporting pillars had been detached from the wall and sunk. Itwas now resolved not to load the piles and relieve the pressure from the destroyed wall as much as possible. This object was attained by the following construction:

The soil just behind the quay wall was dug out at low water and partly under water and the space filled again with crushed bricks. By this means the backing was not only lighter, but the drainage could be carried on at a deeper level.

On the top of these bricks and behind the quay wall another wall was formed of lean béton, which was to take up the whole pressure of the upper backing soil by applying the load of it to the grooved piling and to the piles as low as possible. In order to prevent the stays from being strained by the weight of the new wall and the earth, the ditches in which the stays were bedded were arched, so that lasting provision was made for the drainage of the water.

When the béton wall was finished (it will be seen from the drawing

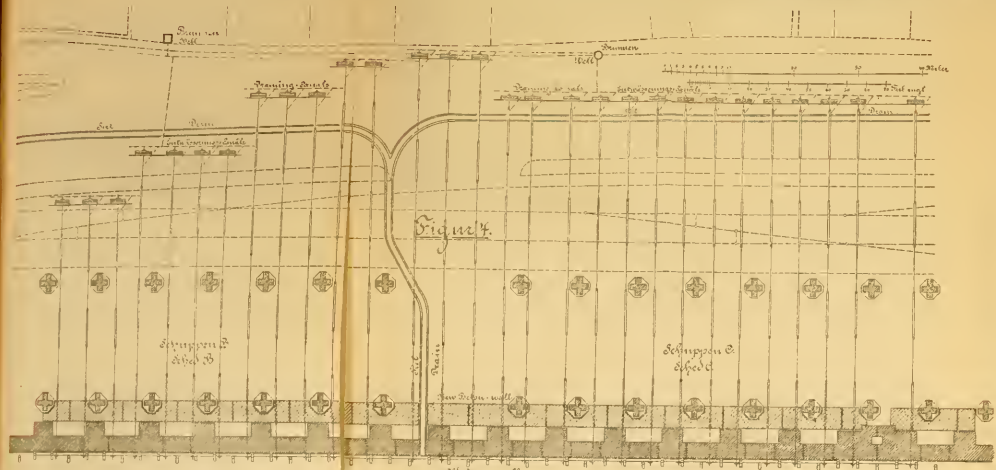
at Fig. 8, *a* and *b*, that the pillars of the sheds projected clear of the profile of the wall), the space behind was filled with brickbats, which could be furnished easily and cheaply from the shelves of the large brickyards on the lower banks of the Elbe. In order to procure a better outlet for the drainage water, earthen pipes were cemented into the béton wall. The front wall was in this way relieved of pressure as far as possible, and had now only to take up the vertical loads, which were produced by the construction of the crane rails, the traveling cranes and the movable loads caused by the railway trucks. The ground for the railway was constructed as follows: the space between the two walls was bridged over by iron I beams, placed about 1.8 m. (5 ft. 10.8 ins.) apart, and these beams were covered with béton covers (caps), so that an empty space was formed between the two walls. This space is accessible from the top, and by means of it the front wall can be constantly observed (see Figs. 3 and 9).

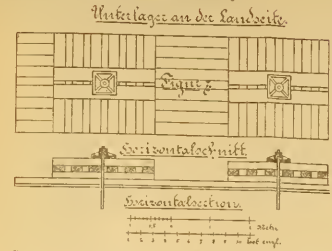
About a year ago the quay was opened to full traffic, and the loads moved just behind the wall and in the warehouses have been exceptionally heavy, but not the slightest change was noticeable in the wall or ground, so that it is expected that the wall has been permanently fixed. The cost of renewing the quays was about 275 000 marks, or 2 000 marks per linear meter (3 ft. 3 4 ins.) of the quay.

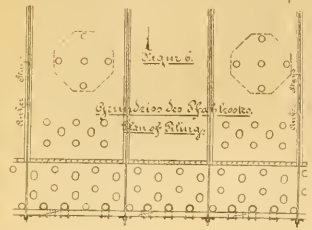
VOL. XXX, No. 648.
TRANS. AM. SOC. CIV. ENGRS.
STAHL ON QUAY WALL AT ALTONA, GERMANY
PLATE I.

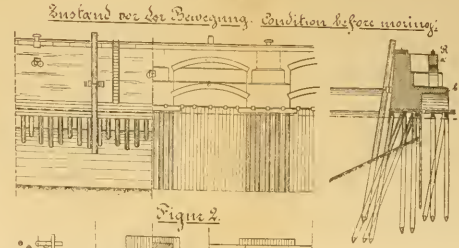
Befestigung des gefährdeten Theiles der Ufermauer am Ostkai.

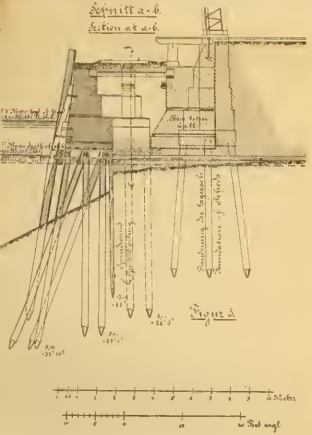
Reinforcing of the endangered part of the wall at the eastern quay.

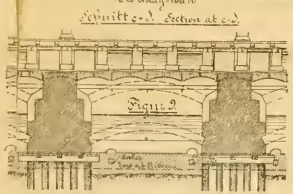


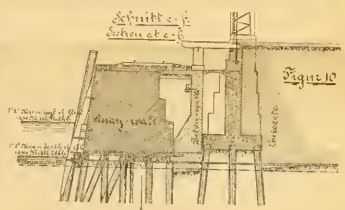


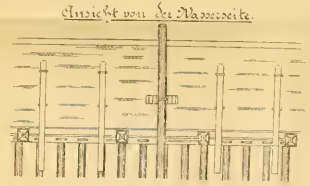


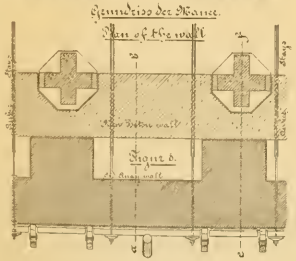


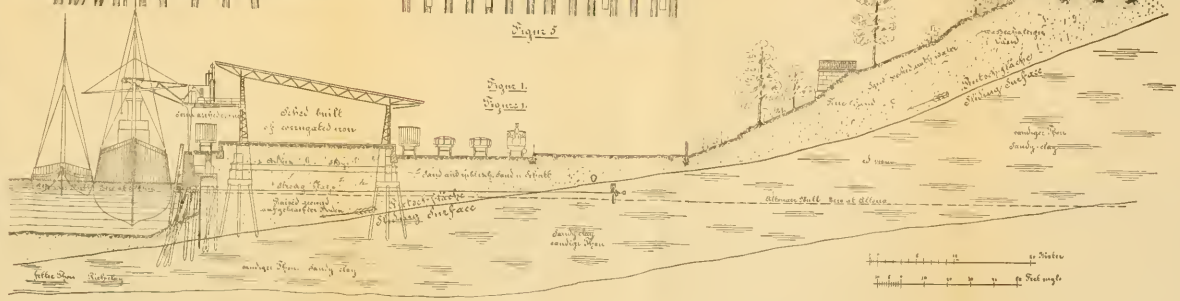












AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

647.

(Vol. XXX.—November, 1893.)

EXPERIENCES HAD DURING THE LAST TWENTY-FIVE YEARS WITH WATER WORKS HAVING AN UNDERGROUND SOURCE OF SUPPLY.

By B. SALBACH, Baurath at Dresden, Saxony, Germany.

Translated from the German by CLEMENS HERSCHEL, M. Am. Soc. C. E.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

The preconceived opinions which are frequently expressed against underground sources of water supply have induced the author to publish the result of his experience gathered during 26 years of practice, devoted almost exclusively to this branch of engineering. Although this paper will consist merely of a series of descriptions of water works of this kind, the attempt will, nevertheless, be made to explain the underlying principles of such works, and it is to be hoped that other gentlemen, active in the same branch of engineering, will add the results of their experience.

An examination of the statistics of water works constructed in recent times will show that the number of such works having under-

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

ground sources of supply has increased in greater proportion than those drawing from rivers and filtering the water thus drawn; and the use of such sources is destined to attain a still greater increase, by reason of methods, which have been discovered quite recently, for changing to a good quality water which, when first drawn, does not seem fit for use. Water of this kind, when drawn from a considerable depth, even when requiring such treatment, offers the best of assurances that it does not contain the germs of destructive epidemics.

Although the experience of the year 1892, in Hamburg and Altona, has shown that a proper filtration of water is accompanied by the best of results in this respect, the assurance is, nevertheless, not present, as was shown in 1893 at the filtering plant of the Insane Asylum at Nietleben, near Halle, that a water containing disease germs will be deprived of them under all circumstances, and that accidental defects in the operation of the filters will not become a source of danger to health. It must also be admitted that a great danger is at hand in the use of underground water, when the wells cannot be kept free from the effects of sewage sinking into the ground, and care must be taken to prevent injury of this kind. It does not, however, argue against such water in general that certain wells have been dug in improper places, and that epidemics have been caused through the infiltration of sewage; it is also an unauthorized assertion, though often made, that the future yield of underground sources of water supply is necessarily a matter of great doubt.

The following descriptions of underground supplies which have proved themselves to be good throughout a long series of years should remove a prejudice of this kind, and, though expectations have not been met in certain places, continuous experience has shown the reason therefor, so that experienced specialists may, in all cases, judge of the existing circumstances with sufficient certainty. In these examples special attention has been given to the development of, and the experience had with, the works of the city of Dresden during 18 years of operation, because especially important points may be illustrated in describing this case, the understanding of which will enable every reader to reach a proper conclusion.

The author was called, in the year 1867, by the city government of Halle to report upon the preliminary works then under way for testing the sources of supply, and upon a project for the water supply of the

city. These works consisted of borings in the Aue, near Beesen, above the junction of the Elster and the Saale, which had penetrated a thick and widely extended bed of gravel, and in drawing for a long time large quantities of water from a well which a chemical analysis had shown to be fit for the proposed supply.

It had been assumed, before undertaking the examinations, that the water had penetrated this bed of gravel from the neighboring rivers, and that when removed the river water would replace it after a natural lateral filtration of the same through the banks. That this assumption was erroneous was speedily shown after a short operation of the pumps at the water works. In the first place, the water taken from the gravel was shown to have an entirely different chemical constituency from that of the river water, and it was shown, furthermore, by a series of experiments that a natural filtration of the river water was wholly impossible.

To prove this, several wells were dug along the shore of the river, in which the ground water maintained a level of about 4 ft. lower than the river water, following a suitable operation of the pumps. When this was the case, the earth over the whole water level on the river side was entirely dry; extending the well laterally until within about 14 ins. of the bank of the river would cause no flow of water from the bed of the river into the well, and this last 14 ins. of soil was dense and impervious to water. These examinations proved that, in this case at least, there was no flow of water into the bed of gravel of the valley from the river, and that the water drawn from the wells was an independent underground source of supply which flowed parallel to the river through the gravel and followed the general slope of the valley.

The opinion expressed at the time was quite general that this bed of gravel constituted an independent basin, which would be exhausted after a certain amount of pumping; but this was disproved by the fact that after pumping a long time, the water in the wells very quickly regained its original level. There was then great difference of opinion and much fear regarding things which have now, after an interval of 26 years, become well known to all experts in this branch of water supply. Nevertheless, the fundamental principles of well-water supply have been established by the experience of the water works of Halle. These works now supply six times the quantity they were originally built to supply, due to the growth of the city and the draft of a large

number of factories which have been erected since that time. The construction of all the larger well-water supplies for cities in Germany has been based upon the experience of Halle.

An examination of the water derived from the gravel showed that it was harder than the water of the adjoining river, while that of the Saale was still harder than the water of the Elster gravel. The water of the Elster gravel was shown, shortly after the pumps were started, to contain considerable iron and to be disposed to breed algæ, which circumstance caused the abandonment of the wells situated in it. Since that time, the Halle works have yielded an unexceptionable well water, up to the amount, in the year 1882, of about 3 750 000 galls. per day (12 840 cu. m. per day).

In passing, the author desires to refer to an attempt, which was made in 1868 at Halle, to purify the water containing iron, and which has quite recently again been used.

It consisted in allowing the water to flow from the main pipe upon a box of gravel, having a sieve at the bottom, and a sand filter beneath it, and in this way it was favorably exposed to the action of the air. The result was a quick oxidation, as well as an escape of the sulphureted hydrogen, and a yield of perfectly clear and beautiful water. This process was adopted at a later date by the management of the Charlottenburg water works, at which place the same evils were encountered, and, subsequently, was improved upon; and ferruginous water of these works, found in certain of the wells near the Devil's Lake, was thereby made of a desirable character.

This process may properly be used in many other places at which the supply of well water has had to be abandoned on account of the amount of iron held in solution; it will be the more serviceable in such cases, because, in most instances, water of this character, so soon as freed from its iron compounds, has been shown to be of unexceptionable quality. Frequently, a mere aeration will cause the heavy iron compounds to be precipitated, and will suffice; as, for example, in the case of the water works supplying Norderney, a watering place on the North Sea. The water which is taken from wells dug in the sand of the island is unfit for use, whether for drinking or for other domestic uses, on account of the quantity of iron held in solution. It is aerated in a tower, in which is a stand-pipe, and, from this, passes to a closed reservoir, in which the iron is precipitated sufficiently during the

slow movement of the water, and is fit for the domestic supply of the island immediately upon leaving the reservoir.

Water containing iron is found in large quantities in the subsoil of the whole North German plain; in Holstein, Schleswig, Friesland, in the eastern part of Prussia, and in Holland, and has hitherto been considered entirely worthless. This water will soon, no doubt, be made use of in the manner described, because, by reason of its being drawn from a considerable depth, it offers the best of assurances of being entirely free from contamination in any form, while a filtration of river water does not offer absolute assurances that such water, when once charged with bacteria, will, under all circumstances, again be cleared of them. Shortly after the completion of the Halle water works, the city of Berlin directed Civil Engineer Veit Meyer to make examinations for a supply of well water from the vicinity of that city. Such examinations were made near the Tegel and the Müggel Lakes, and showed very favorable results, so far as quantity was concerned. But the water drawn from these wells presented the same phenomena as that drawn from the Elster gravel, so that they were not used. Similar phenomena were also observed at other places; as, for example, at Nürnberg, Leipzig, Königsberg in Prussia, and at certain wells of the Charlottenburg water works, as above mentioned.

This quality of the well water of the Weser valley was exhibited in a striking manner in the experiments conducted at Bremen in 1869. A gallery constructed in the vicinity of the Weser, for purposes of preliminary examination, yielded very considerable quantities of water. This, when drawn from the wells, was perfectly clear; but in a very short time became of a brown color, and, after a short period of settling, precipitated a considerable amount of clayey iron oxide, leaving the water above it perfectly clear.

The results found at the water works built by the author for the city of Groningen, in Holland, will be of general interest. Although they do not strictly belong here, it is the author's wish to publish them for what merit there may be in them. These works have their source of supply in a river, called the Drentsche Aa, and water is delivered to the city after settling and a sand filtration.

After heavy rain storms, and during spring freshets in the river, the water is colored brown by turf bogs situated further up stream and retains its yellowish, brownish color after filtration, although other-

wise clear. To remove this color, a small quantity of alum* is added to it by a small pump, while on its way to the settling basins. This addition of a saturated solution of alum amounts to about 1 to 10 000, or 1 to 20 000, of water. In the settling basins the alum is speedily distributed throughout the mass of water and greatly aids its clarification. The coloring matter contained in the water, principally iron, is precipitated, and the water after settling from 12 to 14 hours, and being filtered, becomes entirely clear and without color. A chemical examiuation of the filtered water shows that the alum disappears from the water after clarification, and that the sulphuric acid contained in the water is increased by a minute and hardly perceptible amount. A very important result was proved by the experiments of Professor Dr. von Calcar, of Groningen, viz., that during this operation, every trace of bacterial life had vanished.

The alum which is gathered in the residue of the settling basins retains its action for some time, so that the amount of alum to be added can be diminished. The sludge removed from the settling basins forms a valuable manure. The cost of this operation is very small, in view of the use to which the sludge can be put, and it is of material benefit to clarify the water and free it of iron compounds, and to destroy deleterious germs contained in the water. The system has been tried at this place for 13 years and may, in other like cases, be of great service.

In certain cases, when there is only a small amount of iron in the water, as, for example, at the new water works of the city of Leipzig, it has been left for consumers to remove the same by individual filters; but this cannot be recommended, as it is difficult to control the cleanliness of such filters, and it would be better that the treatment and clarification of the water should take place before it is allowed to enter the distribution pipes.

To deliver water in such large quantities as 10 000 000 to 25 000 000 galls. (40 000 to 100 000 cu. m.) per day, and of such quality as hygiene demands, and, generally, to satisfy the requirements made in recent years upon a public water supply of large cities, is a matter fraught with many difficulties, the more so when it is required that all the water delivered to consumers, whether used for drinking or other household

* Experiments with an injection of a small quantity of chalk produced approximately the same result; but the water in that case was made to have a disagreeable taste.

purposes, or for the watering of streets and gardens, is to be of unexceptionable purity. Such requirements are more easily complied with in the case of small cities; but are, even then, difficult of attainment.

The opinion is often expressed that cities should avoid no expense to supply themselves with water from mountain springs. In discussing the water supply of Hamburg, assumed to have been the vehicle of the cholera epidemic of the year 1892, it was suggested that the city should get a supply from the Harz Mountains, and, previous to this, it had been suggested to supply the city of Berlin from the same source. The author has attempted, during the course of many years, to find enough spring water to supply the smaller cities situated along the foot-hills of the Harz Mountains, but has failed almost completely, and it is now in contemplation to supply these cities by means of artificial storage reservoirs.

Examinations have shown that the Harz Mountains contain but very little spring or well water, so that any plan to supply larger cities from such a source must be based upon total ignorance of the circumstances of the case. There are no extensive drift valleys reaching out into the plain below and constituting the real collectors or reservoirs of ground water, which, flowing off the mountain ridge, enters these drift deposits and pursues its way to the valleys and plains by underground courses. The decomposed granite and slate of the mountain ridge have made the drift foot-hills stanch and almost impervious to water, so that the water, not being able to sink into the ground, runs off on the surface. The foot-hills which consist, for the greater part, of chalk débris, and which dip upwards to the mountain range, yield a water so hard that it is not fit for purposes of a domestic water supply. Similar conditions obtain in the Thüringer Forest, although we find here a greater amount of drift valleys and of deposits of this material in the river valleys, which have their rise in this mountain range.

More copious spring-water formations may be found in the Riesengebirge, in the Eiffel, in the Carpathian Mountains and especially in the Alps. Without exception, we there find, adjoining the mountain range, great valleys of drift formation, which may be followed down into the plain, and in which the water, falling on the mountain range, is mixed with the rain water penetrating the soil of

the valleys and flows as an independent spring-water stream until these gravel and drift formations reach the bottom of the sea. The volume of these subterranean streams may be seen from various phenomena. For example, in the North Sea, in the vicinity of the islands surrounding Holland, there are springs which may be recognized in the water of the ocean, 2 or 3 miles from the shore, and from which fresh water may be drawn apparently from out of the ocean. Similar springs may be found in the Adriatic Sea, near Fiume, Abazzia, Triest, and in other places where the rain water penetrates into the ground in the Karst Mountains, appearing again in the sea, so that at these places the surface of the sea is slightly raised up, and a whirlpool may be observed.

As a surface stream is formed by an immense number of smaller independent contributary streams, and thus gradually becomes a mighty river, in like manner subterranean water-courses are formed from similar subterranean streams. These, gathered together in adjoining pervious sand and gravel beds, form larger streams, which flow along impervious inclined layers, situated beneath those which carry the spring water.

In the plains there are often found a series of strata saturated with water, which are separated by other great strata of clay, on penetrating one of which the water, which is under pressure in the gravel beneath, will rise to the surface and form a spring.

In certain valleys the ground water, flowing along beneath the surface, rises to such a height that it threatens to convert the surface of the ground into a swamp, which can only be prevented by affording a channel in the shape of deep ditches for the flowing off of the spring water. Such artificial springs often yield a very large quantity of water, and indicate localities where still greater amounts may possibly be gathered.

But subterranean streams are fed, not only from the mountain ridges, but at all points where the soil is pervious between them and the surface. Rain falling upon such a surface penetrates the ground in a vertical direction, so long as it does not meet with impervious strata, and adds to the flow of the subterranean streams. The rain water thus penetrating the ground takes up carbonic acid from vegetation and from the surface loam and dissolves with its aid a portion of the minerals found on its way. The more such sub-

stances taken up by the water, the less useful it is for a domestic water supply. If lime be the impurity, hardness results; if salts of iron, the water when exposed to the oxygen of the air, at a later stage, will show the phenomena previously described in this paper; or if the water be vitiated by local deleterious matter, it will have become unfit for purposes of a domestic supply.

To find such subterranean contiguous strata saturated with spring water, and to open them up at proper places, at the same time seeking to know the direction of their flow, their extent and their velocity, constitutes the basis of a successful spring-water supply. Failure has often resulted from lack of sufficient preliminary examination, which has then been the occasion for opponents of this system to represent such water supplies as of dubious value. In giving examples of such works the author desires to say that he distinguishes between cases where water supply works have been wrongly planned and those other cases where the works were at first sufficient for the then requirements and had to be extended on account of the increase of water consumption. In the latter case, it has often been maintained that spring-water supplies can be successful only for a certain length of time, and that a diminution of their yield can only be prevented by continually extending the works.

It has, to be sure, happened that certain works have yielded less water, after being in use for some time, than had originally been assumed. They had to be extended until the demand upon them had been brought into balance with the natural yield of spring water. The yield depends, of course, upon the amount of rainfall, so that considerable fluctuation in the elevation of the ground water may occur, which in certain cases had not been guarded against. Nevertheless, proper preliminary examinations will determine with great certainty what amounts of water may be drawn from wells, even after periods of the greatest drought, and forethought must be taken in such examinations as to the direction in which further extensions may, when necessary, be made. It must also be remembered that the engineer has not always at his command a sufficient appropriation to build works of sufficient capacity to last for all time, and that he can do no more than to provide, to begin with, as to the manner of making extensions.

One of the first spring-water water works in Germany was that of the city of Leipzig. An attempt was made in that case to get the

water for a domestic water supply from two wells of large diameter, which were situated in the valley of the Pleisse, near the shore of the river. Whether at the time it was intended to draw from an independent supply of ground water, or whether a natural filtration of the water of the river was planned, was probably not settled at the time; but the water from these two wells was ground water, and contained a great deal of iron, so that complaints of the consumers speedily followed. Upon this, some of the river water was mixed with the well water. This diminished the proportion of iron in the water. At a later day, collecting galleries were built on the shore, also artificial filters, and through many years the water of the Leipzig water works left much to be desired with respect to its quality and quantity.

Only a few years ago a subterranean stream of water was found in the vicinity of Grimma, near Naunhof, in an old, abandoned channel of the Mulde which, since that time, has supplied Leipzig with an abundant supply of water.

The water works of the city of Bernburg were built in the year 1876. The source of supply consists of a row of wells in the valley of the Saale, which were originally built to supply 625 000 galls. (2 500 cu. m.), per 24 hours for a population of 16 000. It was then extended, as the population had increased to 34 000, on January 1st, 1893, to a capacity of 1 000 000 galls. (4 000 cu. m.) per 24 hours. Until a short time ago, these works yielded an unexceptionable water which remained absolutely clear and pure, even during the time of freshets in the Saale, although the plain in which the wells were dug is overflowed at such times. The variation of temperature was only $\frac{1}{2}^{\circ}$ in the course of the year, not being affected, at the distance the wells were from the river shore, by the warmer river water.

From the result of tests made for the water works for the city of Halle the assumption was justified that this was a case of independent subterranean ground water; but this assumption was disputed when the water from the Bernburg wells similarly with that of the Saale was found to contain salt to a considerable extent. The salt in the Saale has its origin in the discharge from the Mansfeld mines, which is allowed to flow into the river, and it has increased in the course of time to such an extent that the waters of the Saale are, for other purposes, useless. Some 1 750 000 lbs. (82 000 kg.) of salt, are thrown

into the Saale every day, and in low water of the year 1892, the Saale was nearly brine.

Although the water drawn from the wells of the Bernburg works, as well as that drawn from other wells in the valley of the Saale, contains, by no means, the quantity of brine that may be found in the river itself, yet it contains enough to render it unfit for drinking and for such culinary purposes as do not call for the addition of salt.

The city of Bernburg, which hitherto has taken a great deal of pride in its water works, was put in an unpleasant position, being compelled to seek a new water supply, which it is intended to bring to the city from a distance of 5 miles. For this purpose a well has been dug in an extensive bed of gravel, situated between Stassfurt, Rathmannsdorf, Güsten and Ilberstedt. This gravel is from 30 to 40 ft. thick and rests upon an impervious stratum of red clay. The plain has an area of about 200 sq. miles and allows a great mass of rain water to penetrate into it. The water gathers on the impervious red clay and flows, following the inclination of the stratum, towards the valley of the Wipper and the Bode, which are about 3 miles from the well.

This well yielded about 1 500 000 galls. (6 000 cu. m.) in 24 hours, and the yield was shown to be capable of still greater increase. If it were argued that the water of the Bernburg water works had been mixed with a portion of the salt contained in the water of the Saale, on account of natural and only partial filtration of the Saale water, it would have to be admitted that so much of the water of these works as does not come from an independent ground water supply had been successfully filtered for 16 years. But it must be considered that frequent freshets in the Saale had allowed much salt water to penetrate the surrounding soil through which the subterranean water flowed; also, that salt will penetrate layers of earth; as, for example, clay or muddy soil, through which clean water will not flow to any material extent.

The reason for the location in valleys of water works depending on a supply from wells is generally to be found in the situation of gravel and shingle deposits, which, reaching down from the mountain ridges to the valleys and carrying off spring water, are exposed to a greater extent, in the deep cuts formed by the river valleys than elsewhere. The

greater part of these depressions was formed by movements upon the earth's surface a long time before the existence of rivers as they now are, and the deposits of gravel and shingle referred to are of much more ancient origin than the rivers themselves, which latter are generally excavated in more recent formations. Especially interesting examples in this regard are the sources of supply of the water works in the valley of the Rhine, of Bonn, Cologne, Elberfeld, Düsseldorf, Mannheim, and also of the city of Dresden, which will be described more in detail.

In Bonn, water is taken from wells in the vicinity of the Rhine, which yield considerably harder water than that of the Rhine, the former being of exactly the same constituency as the spring water found at a considerable distance from the Rhine.

The chemical analyses made regularly, several times in the course of a year, of the water furnished by the Cologne water works, are exceedingly instructive. As the consumption increases, the level of the water in the well varies with it, being more or less depressed, according to the amount pumped. Chemical analysis shows a softer water the greater the amount pumped. As the works were extended by the construction of new wells, the water became harder until an increase in the amount pumped, upon which it became softer again, and again steadily became harder when new wells were constructed.

This process may easily be explained. As the consumption increased and more water was drawn from the wells than had originally been contemplated, the level of the water was lowered more than usual. Excessive depression of the level of the water in the wells would cause the water from the river to enter the wells, and, by mingling with the independent ground water supply, would make it softer. The construction of new wells would raise the level of the ground water in all of them; the curved surface of the water table, feeding the wells, would no longer intersect the water level of the river, and the result would be a harder well water, until a further increase of consumption would again cause the river water to flow in, and would again result in the wells yielding softer water. Of course, each case must be judged by itself, to determine what level of the water in the well will cause the river water to flow into it, and what level may be depended upon to furnish only spring water, dependent upon the form and extent of the geological formation.

Examples of the yield of large quantities of spring water for many years will counteract the often-expressed prejudice that all such works yield nothing but filtered river water, especially as the votaries of this opinion generally deny the possibility of a natural filtration of river water. The direction of a current of spring water into wells must primarily be considered, in order to get a clear idea of how the wells are fed. The water works of the city of Munich furnished an interesting example of well-water supply. The subsoil of the Bavarian plain, situated to the north of the Alps, consists of a deposit of débris of great depth emanating from the limestone of the foot-hills. Below the foot-hills there are valleys of this débris which adjoin the subsoil of the plain, consisting of gravel and shingle. The strata of the latter have between them an impervious bed of tertiary blue clay of considerable thickness, the surface of which is corrugated and crossed with valleys.

Rain water falling on the mountains, as the surface is very porous, penetrates into this débris and into the gravel beds over the blue clay, and, following the latter, flows through the subsoil until it appears on the surface at certain points, or is forced to run into the river, or into brooks, some of which disappear into the ground after reaching the soil of the Bavarian plain. During the course of the examination made by the author for the supply of the city of Munich, during 1874 and 1875, many such disappearances were discovered, as, for example, the disappearance of the Kirch pond in the vicinity of Holzkirchen, near Mangfall valley, which is fed by the rainfall of the surrounding foot-hills, and whose outflow at one time gauged 18 750 000 galls. (75 000 cu. m.) in 24 hours, but which wholly disappears after flowing about 1 mile in the brook called Hell's Ditch.

As the surface of the impervious layer of clay is corrugated, as has above been noted, large quantities of spring water enter the valleys from the side hills, where the clay has thus been exposed by the cutting action of the streams, sufficiently large, in some cases, to run several mills. It is one of these springs, on the left side of the Mangfall valley, near the village of Darching, which supplies the city of Munich. The water, thus found, is of very good quality—not being too hard—for the domestic supply of the city.

The spring water in the valley of the Inn is much harder than that of the Mangfall valley, and is not fit for a domestic water supply

although large in quantity. That of the valley of the Traun, near Scharlinz, for the supply of the city of Linz, is of excellent quality and of unusual volume.

Not only in the Bavarian Alps may these deposits of débris and and gravel be found, but the whole valley of the Danube, down to the Iron Gate, has been thus filled to a great width and thickness, and the beds join similar ones from the mountains. The water works of the city of Pressburg furnish at all times an unexceptionable supply of spring-water from the deep well which is situated on an island in the Danube, up stream from the city. By lowering the level of this well about 4 ft., the well was caused to yield about 2 500 000 galls. (10 000 cu. m.) in 24 hours. Experiments made by the author in 1890 on the left bank of the Danube, above Buda-Pesth, demonstrated an unlimited yield of well water of the best quality to be had from the gravel of the Danube valley. The yield of each of three experimental wells, by lowering the water level about 10 ft., was from 3 000 000 to 3 500 000 galls. (12 000 to 14 000 cu. m.) in 24 hours, at which yield the water table gently rose from the level of the water in the well to the banks of the river, and it was concluded that a supply of 50 000 000 galls. (200 000 cu. m.) in 24 hours could be drawn from the property of the city. The construction of new water works from this source of supply has since been determined upon. One of these wells, situated about 1¼ miles from the river and yielding about 3 500 000 galls. (14 000 cu. m.) per 24 hours, furnished water of an especially good quality, according to chemical and bacteriological analysis. Mixing with it some water containing iron taken from the left bank of the valley, after standing for some time it became slightly cloudy; this was caused by the effect of the oxygen of the atmosphere and the low form of oxidation of the iron contained in the water. The attempt to mix this water before it had been exposed any length of time to the air, in equal parts with water taken from one of the other wells, resulted in no trace of cloudiness, because the iron salts were no longer present in sufficient quantity to be affected by the oxygen of the air.

Similar results, as regards the subterranean flow of water, were found during the construction of the water works in Mahrisch-Schönberg, in the valley of the Theiss, and at Olmütz, in the valley of the March; but, in the latter case, the river-bed was found to consist, for a depth of from 26 to 33 ft., down to the underlying blue clay, of fine

silted sand, in which very little water of a pure quality could be found. It was necessary to go about 1¼ miles from the river, to find the water-bearing gravel bed, lying above a layer of blue clay, some 40 odd ft. below the surface. This gravel, originally coming from the Glatz Mountains, contained water of a most excellent quality, and of such volume that, in constructing the foundations for the pump-house, some 6 000 000 galls. were pumped from the pit every 24 hours in the effort to lower the ground-water level about 7 ft.

Large volumes of ground-water are found in the great masses of gravel filling the subterranean valley of the Ruhr, and a number of water works, as, for example, those of the cities of Dortmund, Mühlheim, Oberhausen, Barmen, etc., and the very extensive water works of Krupp's Steel Works, have been using this water for many years. It differs materially, both in quality and clearness, from the water of the Ruhr itself.

The water works of the city of Cöthen are 7½ miles from the nearest water-courses—the Mulde and the Saale. During the preliminary works conducted by the author in the year 1887, beds of water-bearing gravel, 26 to 33 ft. thick, were found in the vicinity, lying above a stratum of blue clay and under a thick upper covering of clay. The examinations showed that the surface of the blue clay had in it several deep grooves which apparently converged toward the Peter Mountains. These grooves contain the water-bearing gravel beds, and a well dug in one of them yielded 1 500 000 galls. in 24 hours of the purest and most serviceable water for domestic supply. When yielding this amount, the capacity of the well was not reached, as a depth of about 20 ft. of water still remained in it.

Attempts to get well water in the valley of the Theiss and Maross, and throughout the valley of the Oder, have failed. In these valleys, nothing but fine silty sand and clay were met with, and, besides, the little water found in the upper Oder valley, in Austrian Silesia, was strongly impregnated with iron.

Further examples of successful wells may be found in the water works built by the author in the years 1875 and 1876 for the city of Crefeld. The occasion for building these works was found, not only in the limited supply of the wells of the city, but also in the poor quality of the water, which had been vitiated by the infiltration of sewage from adjoining buildings. The supply was sought in a region

of country not yet affected by sewage. In doing this, it was necessary to determine the direction in which the subterranean water-courses were flowing, so that the new source of supply could be placed up stream from the city as regards these underground currents.

The tract of land contemplated was situated some 6 miles from the Rhine, and proved to consist of a bed of water-bearing gravel about 36 ft. thick under a top layer of clay, from 3 to 6 ft. thick; under the gravel, a stratum of clay some 6 ft. thick and then another bed of gravel which was explored to a depth of 90 ft. Borings were not continued at the time; but later borings in the vicinity of the works showed blue clay at a depth of 116 ft., rising towards the Rhine, and appearing in its bed in some places. This stratum of clay forms a deep channel in which flow underground streams of water of great volume. In the preliminary examinations, samples of water were taken, both from the upper and lower beds of gravel above referred to. Chemical analysis showed a small percentage of iron in the water of the lower gravel bed; but, nevertheless, the water of both gravel beds was declared to be of a good and wholesome quality.

The works constructed on the basis of this opinion, and on the quantitative experiments, yielded a very fine water. They were built of a capacity of 2 000 000 galls. per day, with the expectation of increasing this quantity in the future by deep wells to be bored into the débris lying under the bed of gravel.

After 10 years' service, the city of Crefeld having grown very largely, and the consumption having increased to 1 500 000 galls. in 24 hours, these deep wells were put down. It appeared, however, that the water lost materially in quality, on account of the iron contained in the water taken from the deep wells, and these were accordingly again closed. In the meantime, on account of the sewerage works of the city of Crefeld, and especially on account of the deep sewage channel leading to the Rhine, which had given the ground water of the vicinity ready means for flowing off, and had lowered the water-table in the vicinity of the city by about 4 ft., the upper gravel bed no longer yielded so much as 1 500 000 galls. per 24 hours.

Inasmuch as it was expected that the consumption of water would soon demand a larger capacity, renewed experimental borings were undertaken in the direction of Gladbach, which resulted in the discovery at this place of a bed of gravel 116 ft. thick, without any inter-

vening clay. A well sunk to a depth of 116 ft., and taking water only at this depth, yielded water of good quality, but it contained the same quantity of iron as did the water of the deep wells of the existing water works. Continued experiments led to the result that at this place water free from iron could be found at a depth of from 60 to 66 ft., and this result was confirmed by putting down two wells to a depth of 60 ft. on the same piece of property, which have yielded for years an unexceptionable quality of water in abundant quantity, so that these works supplied, in the year 1892, up to 3 750 000 galls. per 24 hours. In the meantime the author built a second water-works plant in Crefeld, of a capacity of 7 500 000 galls., and the last two years have shown that from the nine wells of these works water of sufficient quantity and of proper quality can be drawn at all times.

Artesian wells in Silesia, at Breslau, Oppeln, Laband, Peiskretscham, Gleiwitz, etc., have been bored with considerable success. The author had charge, for example, in the year 1875, under orders of the Royal German Government, of the borings of artesian wells near Zawada, in Upper Silesia, in which sandstone was struck at a depth of about 500 ft., under a thick bed of chalk, and which yielded, by means of pumping, water of very good quality up to about 4 500 000 galls. in 24 hours. The project of supplying water from these wells to the Upper Silesian manufactories not having received further attention, the drought of the year 1892 forced a consideration of such a supply. Gaugings taken in 1892 showed a yield of about 3 000 000 galls. in 24 hours. But it is to be assumed that during the course of 17 years the lower portions of the pipe have gradually silted up, and that a redrilling and cleaning of the bore hole will restore the former yield. It was formerly projected to drill a well of still larger diameter, in which the level of the water could be lowered by pumping, and thus cause a still greater yield. From experiments made by drawing the water down, and noting the yield at different depressions of the water in the well, it was at the time determined that by lowering the water-table 20 ft. this well would have a yield of 8 250 000 galls. per 24 hours.

The water works of the city of Dresden were designed and built in the years from 1873 to 1875, of a capacity of 7 500 000 galls. (30 000 cu. m.) per 24 hours. In the meantime they have not only yielded the desired quantity and quality of water, but have gone beyond it. The

maximum daily consumption during the last 11 years has been as follows :

Year.	Month and Day.	Million Gallons.	Cubic Meters.
1882.....	July 20th	6 669 000	26 676
1883	" 5th	7 342 000	29 368
1884.....	" 17th	7 047 000	28 188
1885.....	" 27th	7 848 000	31 392
1886.....	May 22d	7 730 000	30 920
1887.....	July 30th	8 708 000	34 832
1888.....	June 25th	8 836 000	35 344
1889.....	" 8th	9 230 000	36 920
1890.....	May 24th	8 762 000	35 048
1891.....	Aug. 29th	8 506 000	34 024
1892.....	" 20th	10 561 000	42 244

The analysis of a sample of water taken from the distribution pipes, made by the Government Chemical Laboratory, on August 12th, 1891, being at a time of maximum consumption, gave the following result in 1 liter of water (1 000 gr.) :

Grams.	Substances.
0.1569.....	Solids held in solution in which were contained :
0.0020.....	Organic substances.
0.0113.....	Chlorine.
0.0096.....	Nitric acid.
0.0422.....	Chalk.
0.0124.....	Magnesia.
0.0000.....	No trace of the nitrous.
0.0000.....	No trace of ammonia.
5.96.....	German degrees of hardness.

Professor Dr. Neelsen reports as follows upon the bacteriological examination of the water, made in the year 1892.

"In accordance with a request made on September 15th of the present year, the undersigned, in conjunction with City District Physician Dr. Niedner, and Government Engineer Krumhaar, took samples of water at the places named below, on October 8th, in the afternoon, between 3 and 6 o'clock. The samples were gathered in sterilized glass jars, and the bacteriological examinations were commenced the same evening by pouring some of the samples upon gela-

tine plates. Of each sample, three plates were prepared with ½ cu. cm. of water each. The number of developed germs on these plates was determined after 36 to 60 hours, and, from the average number found, the round number of germs in each sample per cubic centimeter was determined. At the same time, the number of species of bacteria on each plate was determined by inspection, as far as possible. (An accurate count would have taken a good deal of time, and would have been of no material advantage.) Finally, the several species were propagated on gelatine, in order to determine their identity with known forms of bacteria.

"The result was as follows, for the several samples taken:

"I.—Water of the City Distribution Pipes.

"(1) Water taken from the well at the pump house contained from 70 to 80 healthy germs per cubic centimeter. In determining the species, it was shown that approximately half of all the colonies that had been developed were formed by *Micrococcus aquatilis* (a circumstance which was repeated with all the water samples, and, therefore, need not be mentioned again each time). About one-third of all the colonies were composed of two organisms, in about equal number, viz., *Micrococcus flavus desidens* and the *Bacillus fluorescens simplex*. The remainer, about one-sixth, was chiefly composed of colonies similar to a species of yeast, which slowly dissolved the gelatine, and of the *Bacillus fluorescens liquefaciens*, as well as of a few colonies of the *Bacillus subtilis*.

"(2) The Fischhauser Revier basin contained 80 to 90 germs to the cubic centimeter, and of the same sort as the well, and also in approximately the same proportion as to numbers. The increase in the number of germs may be explained by the circumstance that in this case about the same amount of the *Bacillus liquefaciens simplex* was found, in addition to the above-named *Bacillus fluorescens liquefaciens*. No specimens of *Bacillus subtilis* were found in this sample.

"(3) The same favorable condition as regards the absolute number of germs, as well as regards the species found in the case of the two above examinations, was maintained in the bacteriological examination of the further five samples taken from the whole distribution system.

"Although the number of germs was somewhat larger than in the well and in the collecting basin, there was only a slight increase. In the kitchen faucet of the dwelling house of No. 33 Wincklemann Street, there were about 100 per cubic centimeter; approximately, the same number in the stand-pipe at the Bohemian Railway Station; in the kitchen faucet of the dwellings No. 72 Bautzner Street, second story, and No. 20 Blochmann Street, third floor, 100 to 120; there were from 120 to 150 in the faucet of the microscopic laboratory of the City Hospital. The number of species present remained almost the same. In three samples (Wincklemann Street, Bautzner Street and the City

Hospital) single colonies of *Bacillus aquatilis sulvatus*, and of *Bacillus liquidus* were found, besides the species already named, and these were probably not found in the well and in the basin by accident, because they are usually met with in all water. Detached colonies of *Micrococcus candicous*, of yellow sarvine, and of little brown *cocci*, could be discerned, though probably they came from the air.

"The increase in the number of bacteria in the water taken from the distribution pipes does not indicate a growth of the germs in the pipe. The growth of the germs takes place on the damp walls of the faucets, on which the bacteria find favorable conditions of development when in contact with the air. This is shown by the disproportionate increase of the species which thrive upon exposure to the air; that is to say, of the *Bacillus liquefaciens* and the *Bacillus fluorescens liquefaciens*. Colonies of this kind are found in greater numbers, and, in this way, contribute to the increase of the total number of germs found, the number of other germs showing no increase.

"The effect of the orifice of efflux, especially when its interior surface had not been cleaned for a long time, was clearly shown in the case of the stand-pipe of the Bohemian Railway depot. At the preliminary examination on September 20th, it was found that a number of germs at this stand-pipe were twice as great as in the kitchen faucet of the dwelling house No. 1 Stall Street, on the ground floor, the number of germs in which agreed perfectly with those found in the kitchen faucet of the other houses. After cleaning the stand-pipe at the depot in a thorough manner, the sample taken on October 8th gave the result above noted, agreeing with the number of germs taken from the other faucets.

"Inasmuch as all the bacteria found in the water taken from the distribution pipes are known to be wholly harmless, and as the number of species and the total number of germs found is a very small one, the water may be declared to be an especially good one, and responding to all the demands of hygiene to be put to it from a bacteriological standpoint."

Although the maximum consumption above given was used as a basis for the construction of the works, it has been exceeded daily, and the demand has been met by the works without difficulty; the question had, nevertheless, to be considered what steps to take to prepare for an extension of the works, in view of the probable great increase in the number of inhabitants to be supplied because of the annexation of certain suburbs.

At the time the works were constructed the city of Dresden had 177 500 inhabitants, which had increased by January 1st, 1893, to 305 144. In view of this increase, the city government has undertaken,

in the course of the last few years, under direction of the builder of the works, a series of examinations, to determine the amount the existing works could possibly supply, and what measures should be taken, to enable them to satisfy a still further increase of consumption. These examinations showed that the existing source of supply could be depended upon to supply 15 000 000 galls. in 24 hours, on condition that the water level in the collecting well, into which the horizontal collecting galleries empty, is lowered deeper than had originally been intended, or had, up to the present time, been necessary. The existing pumping capacity was, however, only 11 250 000 to 12 500 000 galls. in 24 hours. This capacity would, therefore, have had to be increased to 15 000 000 galls. in 24 hours. On the advice of the author, the city government did not adopt this method of extension, as he had shown that an inordinate lowering of the water-table, for the purpose of getting materially larger quantities of water, would probably interfere with the yield of the independent ground-water supply; that the extension of the water-table to a greater distance would cause it to intersect the surface of the neighboring river, and would thus produce a head, or water pressure, which would cause a flow of the water of the Elbe into the subsoil of the river-bed.

In proof of this, the author presented at a meeting of the city government a report of the examinations that had been made up to that time, and added to it the result of observations during the 18 years of operation of the works, which are here given in abstract. The preliminary examination for the construction of the existing works commenced with the examination of the geological situation, aided by the valuable experience of Professor Dr. Geinitz, of Dresden. Further, by a series of borings at the side of the river bank, the level of the water-table in the valley of the Elbe, the formation of the ground in the water-bearing upper subterranean layers and the general constituency of the samples of water taken from the different bore holes, could be determined. The several bore holes put down on the right bank of the Elbe showed a steep slope of the water-table at right angles to the river. The examination of the ground-water taken from these bore holes showed it to be exceedingly pure, and of an appreciably less degree of hardness than the water of the Elbe.

Borings on the left bank of the Elbe showed a less slope of the water-table at right angles to the river. The samples of the

water taken from these borings was also very pure, but was of somewhat greater hardness than the water of the Elbe. Some borings put down in the bed of the Elbe itself, to a depth of 23 to 26 ft., showed that the ground-water in these bore holes would rise some 6 ins. above the level of the river. An examination of the samples of the water taken from these last-named bore holes showed a quality of water, which, like the samples taken on the right bank, was softer than the Elbe water, and of the same constituency as that from the bore holes first named.

The valley of the Elbe forms a wide basin between Pirna and Meissen. This basin begins up stream from Pirna at the point where the Elbe flows out from between the spurs of the sandstone mountains of Saxonian Switzerland. This valley expands to a width of several miles and is narrowed again near Meissen by the granite formations occurring at this place. Down stream from Pirna, the right and left sides of the valley forming the boundaries of this basin are composed of granite and syenite. The granite formation extends throughout the valley up to the contraction of the valley near Meissen. Further down stream the left side of the valley is formed of great layers of chalk, which extend to the vicinity of Meissen, changing to granite only about $\frac{1}{8}$ of a mile up stream from Meissen.

The geological shape of this valley is due to the granite formations on each side, in which there is at this place a deep fault. Some layers of chalk, and of sandstone and chalk, have remained on the sides of the valley, after the destruction of these more recent formations. The geological profile drawn at right angles to the valley near Dresden shows granite on the right side rising from the valley, which on the right river shore is generally met with at a depth of about 26 ft., and rises towards the plateau through the Dresden meadow, and is there covered to a large extent with great layers of fine sand. On the left side of the valley chalk is met with, superimposed upon the granite.

Under the middle of the valley, at a depth of from 50 to 53 ft. beneath the surface, a great bed of clay is found on top of the granite and on top of the chalk, having its origin in the decomposition of the chalk, and which may be considered as a lake deposit. Over the clay the whole valley is filled up with a bed of diluvial gravel mixed with sharp clean sandstone, to the thickness of about 30 ft. and becoming thinner towards the edge of the valley, in which the gathered ground-

water moves in the direction of the slope of the valley. Upon these gravel beds we find thinner beds of sand and clay, mixed with many sharp-cornered boulders of basalt, into which the river-bed has been excavated.

The layer of gravel over the clay is mixed in its lower stratum with boulders up to 800 lbs. in weight, and does not come originally from the mountain formation situated on the edge of the Dresden valley, but is composed for the greater part of basalt and porphyry from the Bohemian Mountains, and of granite from the Riesengebirge, which have been brought down by mighty currents whose course has broken into the valley of the Elbe through the sandstone mountains of the Bohemian and Saxonian Switzerland, and which have been deposited in the valley between Pirna and Meissen. The gravel and boulders found at a depth of from 20 to 50 ft. beneath the surface show by their rounded form what a great way they have traveled before being deposited; while the boulders found in the alluvial soil, in the clay and sand, consist of sandstone or basalt which have been deposited at a later period by the high water of the Elbe, and therefore present sharp, square corners.

It may, therefore, be assumed, as has also been proved by further examination in the upper part of the valley, that the whole valley consists of a channel filled with deep water-bearing layers of gravel ; and that a connection may be assumed between these subterranean layers, and the layers of gravel found in the side valleys, so that the ground-water flowing on the pervious strata in these side valleys will flow from all parts into the Elbe valley, and will unite with the ground-water flowing off in this channel.

This flow of ground-water was discovered, not only during the preliminary examinations made in behalf of the water works of Dresden, but the same formation was shown to exist, by borings and wells constructed throughout this valley, at Radebeul, Ober and Nieder Loosnitz, as well as at the down-stream limit of this valley at Sornewitz and Cologne on the Elbe. At the last-named place, wells were constructed which are situated several miles from the Elbe, and which yield, by lowering the level of the water about 6 ft., from 750 000 to 1 000 000 galls. of excellent water in 24 hours.

The same phenomena may be found in the great valley of the Elbe which is situated above the entrance of the Elbe into Bohemian and

Saxonian Switzerland, at Leitmeritz, and at the junction of the Eger with the Elbe. The Elbe and the Eger flow through this valley; but the Elbe has been crowded by the inclination of the surface of the ground to the extreme right edge of the valley near Leitmeritz, where the hillsides consist of chalk formations with some veins of basalt and porphyry. For this reason the subterranean layers next the sides of the valley yield a hard water unfit for use. The chalk formations dip towards the valley of the Elbe, and the inflow from them into the Elbe, may be plainly seen during times of low water. Water found by means of borings on islands in the river at this point shows the same quality. It is necessary to go to the left shore of the Elbe and somewhat back of it, to get ground-water from the great beds of gravel of the diluvial formation flowing down stream and not yet deteriorated by admixture of hard water. Similar formations have been found in the upper valley of the Elbe, above the junction of the Moldau, near Alt-Bunzlau and Brandis.

In the lower strata of the Elbe valley at this point, great masses of débris from the Riesengebirge are found which may be recognized by their round, polished, red granite and porphyry boulders. An examination has also shown a strong underground flow of soft and very clear water from the sandstone of the Iser Mountains.

It is a remarkable phenomenon to find in almost the whole valley that the ground-water, when not changed by local lateral springs on the edges of the valley, is softer than the water of the Elbe itself. An explanation of this circumstance may be found in the fact that the most copious springs come to the valley from deep-lying and thick strata of granite, porphyry and sandstone; while the harder water, coming from higher and flatter lying strata of chalk flows into the river itself.

The sandstone formation in Bohemian and Saxonian Switzerland yields a harder water than the granite and porphyry formations, on account of the chalk contained in the limestone, because the valleys in it are cut through the upper sandstone and deep into the chalk beneath. In these deep cuts, débris of an alluvial origin has been deposited, in which the ground water coming down from Bohemia flows, and which may be found by deep borings in the Elbe itself, while the channel of the river has been excavated in the sand and clay layers above it.

After the junction of the Moldau with the Elbe, the latter receives

many large inflows of hard water from the chalk formations on the right bank as far as Leitmeritz. The contrary case has been observed in the valley of the Moldau, which, coming from a granite formation, has a very soft water. A short distance up stream from Prague, it enters a slate formation, and at this point the bed is composed of a layer of mixed granite and slate débris. An attempt to gather river water by natural filtration from wells and collecting galleries built parallel to the banks of the river failed, because the water drawn was ground-water flowing parallel to the banks of the river in the slate formation.

This hard water of the valley of the Moldau does not reach the subterranean layers of the valley of the Elbe because it is dammed up by layers of slate ; but it reaches the Elbe itself by being forced to flow up into the bed of the Moldau and becoming a part of this stream. For this reason, also, the water of the Moldau, at its junction with the Elbe, is materially harder than it is near Prague.

The direction of the flow of the ground-water in the valley of the Elbe, between Pirna and Meissen, follows in general the direction of the slope of the valley ; but its velocity is not uniform. The velocity of such streams may be assumed to be greater the more pervious the strata are in which they flow.

The flow takes place in the direction of greatest fall in the strata, and any increase in the amount of inflow, or greater compactness of the strata through which the flow passes, will elevate the water-table of the locality where such changes occur. An accurate examination of the strata must therefore precede any conclusions to be drawn as to the cause of a material elevation of the water-table adjoining lateral valleys. Had a conclusion been drawn from the 35 bore holes put down during the preliminary works for the second water-plant, and from the water-table found by means of these bore holes, as to what the general direction of the flow of the ground-water was, it would have led to the erroneous assumption that this portion of the flow was almost at right angles to the river.

The main current of ground-water in that portion of the Elbe valley down stream from Pirna is in a tolerably straight line, parallel to the valley of the Elbe. It takes up considerable tributaries from the granite mountains and from the Gottleuba valley near Pirna. This runs upon the right bank of the Elbe near Pillnitz and leaves it near

Übigau, crossing under the river Elbe. It flows along the granite slopes of Ober and Nieder Lossnitz, then, turning sharply towards the Elbe, enters its narrow valley near Sornewitz, where it is narrowed in between the granite Spar Mountains and the granite on the left shore of the Elbe, near Meissen. To the north, the valley and the stream of ground-water are cut off by the foot-hills near Bohnitzsch and Grobern, which are pierced by the tunnel of the Dresden and Leipzig Railway, near Niederau. In the valley between these foot-hills and the Spar Mountains a protrusion of granite makes a dam, which stops the flow of the ground-water and forces it towards the Elbe, near Sörnewitz. A short distance above this turn the preliminary works for the supply of the village of Cologne disclosed a water-bearing bed of gravel 60 ft. thick, in which the ground-water stood 4 ft. higher than in the river, less than $\frac{1}{2}$ mile distant.

The foregoing is a full explanation of the geological and ground-water situation of the valley of the Elbe, all of which was necessary to a full understanding of the underlying principles of this kind of water supply, as illustrated by the supply of the city of Dresden. In addition it was known that many sandbanks are found in the bed of the Elbe, which are moved about during freshets. The occasion of such banks is often the entering of ground-water springs which loosen the sand in the channels of rivers, so that the current may readily move it. Such points are easily recognized by the experienced eye, and may be proven by measurements of temperature, especially when there is in winter either a thin sheet of ice or no ice at all next to them.

It was natural to plan, therefore, to take only so much ground-water from the well as was contributed directly to it, and without allowing the water of the river to flow in. The extent of the springs coming from the hills down the right slope of the valley was best shown in constructing the foundations of the lower pump-house. At a depth of excavation of about 6 ft. below the water level of the river, these springs yielded, by means of several steam pumps, about 3 000-000 galls. of water in 24 hours, the water from the side of the river flowing in at the same time in very small quantities.

To gather these side springs, a collecting gallery, about 570 ft. long, was constructed between the right side of the valley and the river. It was placed at a level, so as to cause the water to come up through the most pervious strata of the subsoil, lying above the

granite ledge. It was also planned that the level of the water in the wells, in which the collecting gallery empties, was to be lowered only so deep that the water-table extending from the well toward the river Rhine should not intersect the river below its water level. Even if this were exceeded, and if the water-table would intersect the water level of the river, the river water would flow into the underground strata only upon causing a pressure sufficient to overcome the imperviousness of the fine sand and silt on the sides of the bed of the river, and lying between it and the coarse gravel in which the collecting galleries are built.

As the water level of the river falls, this situation becomes more favorable. The underground springs whose water-table is fixed by the level of the river, flow at a velocity due to the increased head into the gravel beds of the valley ; the flow of spring water upwards into the Elbe through its bed is also greater, and the possibility of intercepting independent ground-water is correspondingly greater and more likely than an infiltration of river water. Circumstances are different when the river level rises rapidly. The water-table cannot rise with equal quickness, but is, after a few days, dammed up by the freshet level of the river. During this damming up, and until its conclusion, the water of the river will penetrate the subterranean layers of the shores, so long as the same are not saturated with spring water, or in so far as it is not kept out by impervious layers of soil. This wave of dammed-up water, in case of a small increase of the river level, is often at a higher level than that of the river, and the river water cannot penetrate very deeply into the soil.

When the collecting galleries are heavily drawn upon, the surface of the water-table is, of course, lower than the level of the water in the river, and greater heads obtain which might force the river water to penetrate the subterranean layers of the river channel. It appears, however, that during continued high water the indications from which a penetration of filtered Elbe water into the ground-water could be inferred obtain only during the first few days ; and even during long periods of high water (in 1876, 120 days) these indications disappeared completely after three or four days, and the normal conditions again obtained. For a long time it was supposed that the fine opalescence, which appears during the first few days of a quick-rising high water, was due to the same ; but these indications appear so seldom, and for

so short a space of time, that they were deemed of no importance. Closer examinations, instituted within the last five years, have shown this slight opalescence to be caused by an infiltration of Elbe water entering through detached porous places of the masonry lining of the well, and the effect disappeared as soon as the ground-water rose in due proportion to the high-water level. This opalescence has disappeared almost entirely since the masonry of the wall was uncovered in the year 1891 and carefully cemented, and it is to be expected that upon the completion of the work, which can only be done during the summer, and at very low stages of the river, it will disappear entirely.

On account of the short distance of the steep hills from the shore of the Elbe on the right bank of the river, collecting galleries could not be placed a sufficient distance from the river to bring them outside the region of overflow by high water, and considerations of river conservancies did not permit water-tight well masonry to be placed above the high water-line, or to raise up the site of the collecting gallery, which is under water whenever the river is 5 ft. above zero of the Dresden river gauge.

The water of the Elbe and the Moldau carries, in times of rising water, a great quantity of seepage from layers of turf in the upper regions of the river, which give it a yellowish brown color, due to the iron and to the acid in the turf, which filtration fails to remove. Hence, in the first few days after the beginning of a quick rise of the river, the water distributed in Dresden will show a slight yellowish discoloration, which disappears as soon as the ground-water has been dammed up. This shows that during this short time filtered river water has penetrated into the subsoil and into the ground-water.

Chemical analyses, regularly instituted during times of high water, has shown that the good quality and clearness of the Dresden water does not suffer during continued high water of the river. The continued analyses also by the Government Chemical Central Laboratory, which has admittedly undertaken the work of proving by its labors that the water delivered by the Dresden water works is merely filtered Elbe water, have shown, and the Laboratory has been obliged to confess, that this proof cannot be furnished. The fact that the water of the Dresden water works is subject to considerable fluctuations of temperature has given rise frequently to the assumption on the

part of those not acquainted with the facts of the case, that for this reason the water may be taken to be merely filtered Elbe water. Figures giving the maximum and minimum temperature are, however, frequently given undue weight, because great fluctuations of temperature are due to the special inflow of high water, and disappear quickly with the high water that caused it. If the collecting galleries were far enough distant from the bed of the river, and beyond the reach of inundation, only a very small variation of temperature would be noticeable. But, even during the first few days of the quickly rising water from the river, during which time the effect of the Elbe water must be admitted, the temperature of the water in the pipes does not approximate to that of the river, which would have to be the case when drawing from 7 500 000 to 10 000 000 galls. per day for months at a time, if the ground-water did not exert an influence of its own upon this temperature.

In January, 1893, the water of the distribution pipes had a temperature of 47° Fahr. (8.2 Cent.) at a time when the adjoining portion of the river was covered with ice, and its bed, also, was in part covered with ice. The cause of the variation of the water in the pipes is to be found in the great effect of the neighboring river as a big reservoir of heat upon the water following the subsoil, and numerous investigations have shown that such variations of temperature occur, also, in the water of wells which are near the river, but the water level of which is higher than the water level of the river.

After consideration of all the circumstances, the builder of these water works advised that, having due regard to the preservation of the superior clearness of the water derived from them up to date, the amount pumped should not exceed 7 500 000 to 10 000 000 galls. in 24 hours, and that the construction of the works, as originally built, did not have for its object the lowering of the ground-water level to extreme limits. To enlarge the water works in the most rational manner, recourse should be had to finding a site on the left shore of the river, in which ground-water of good quality and in sufficient quantity could be found for a second water-works plant. By placing another pumping station on the left bank of the Elbe, the distribution of water, and the assurance of a proper capacity of the whole system, would be obtained in a very advantageous manner.

The principal portion of the city, the old city, has been supplied

hitherto from works through two pipes of 24 and 18 ins. diameter respectively, laid on the Augustus Bridge, and through two 12-in. pipes laid on the Albert Bridge. The pressure supplying the water from the high-service reservoir is that due to the consumption which was assumed when the distribution system was built. No complaints about lack of pressure have yet been made, even in the parts most distant from the high-service reservoir, in the south-western wards of the city, and even when the consumption has reached 10 000 000 galls. in 24 hours. But circumstances would be entirely different if the present pumping station should be enlarged, because the distribution pipes would then not be able to convey a larger quantity of water, or would not be able to deliver it to the consumers under a proper pressure. In that case, special mains would have to be laid from the high-service reservoir over the bridge and down into the heart of the old city, in order that the proper distribution of the larger amount of water could take place.

By building a new pumping station, the greater part of the city of Dresden, the old city, situated on the left bank of the Elbe, will become independent of the present water-works plant and of the mains crossing the bridges, and there will be a great deal more pressure in the rapidly growing old city than could possibly have been obtained by an extension of the existing water works.

The site for placing the new pumping station was chosen for various reasons, somewhat further up stream, between the suburb of Blasewitz and the town of Tolkewitz, on the Tolkewitz meadows, rather than directly opposite the present water works. This was done partly, to get beyond the reach of the rapidly growing town of Blasewitz and to go up stream from the newly located cemeteries, and partly on account of the quality of the water found in bore holes on the left bank of the Elbe. As has already been mentioned, the borings directly opposite the present water works yielded a ground-water which was a few degrees harder than the water of the Elbe. This greater hardness, due to the influx of the water of lateral springs from the chalk formations of the left side of the valley, diminishes as one goes up stream. A well dug for the Court Brewery in Cotta, on the immediate banks of the Elbe, yielded a materially harder water than that of the river itself, when about 40 000 galls. in 24 hours were pumped from it.

The amount of hardness of the water on the left side of the river may be seen by the following figures:

Water from—	Degrees of Hardness.
The existing water works	7.5
The Elbe	12.0 to 13.0
The bore holes in Blasewitz, on the left bank of the Elbe, opposite the present water works	14.0
The well of the Court Brewery in Cotta	22.0
The well in the "Old Market," in the old city, Dresden	35.0 to 36.0
The well on Vienna Street	40.0 to 41.0
The well on Berg Street	56.0
The well of the new water works	10.0
The well in Tolkewitz	10.0
The well in Blasewitz, according to the depth of the well	10.0 to 12.0 to 16.0

An extended series of examinations of this sort demonstrated that the hard ground-water flowing from the left side towards the middle valley of the Elbe and towards the river, did not flow in a direction at right angles with the center of the valley, but went under the old city of Dresden, and reached the channel of the Elbe, down stream from the same.

The fact that the hardness of the well water in Blasewitz is different in different wells is explained by these wells having different depths. The subsoil in Blasewitz consists of a stratum of sand some 26 ft. thick at the surface, then one of clay 6 to 10 ft. thick, and, under this, the water-bearing gravel of the Elbe valley.

The wells dug in the principal layer of sand, and ending above the layer of clay, have a harder water, while the wells reaching into the gravel beneath the layer of clay yield a softer water. From these examinations, it was shown that the proposed site of the second water-works will furnish partly the water of the underground strata of the Elbe valley, and partly the lateral tributaries, coming in from the chalk, and in an easterly direction from the granite and syenite formations on the left side of the valley. Upon this, 35 borings were put down, which showed a very uniform formation of the subsoil, from which the inference was drawn that the same formation extends over a still greater portion of the left Elbe valley.

The surface of the proposed site, situated on an average of from 13 to 16 ft. above the zero of the river gauge at Dresden, consists of meadow and farm lands, in which is found a clayey sand about 13 ft. thick and so impervious that the ground-water from beneath rises some 10 ft. high in the bore holes after these layers have been penetrated. Under these impervious layers is found sand, which, at a greater depth, is mixed with gravel and boulders, until an impervious layer of blue clay is reached, at a depth of 50 to 53 ft.

Chemical analyses was made of the water derived from the bore holes; and these showing a sufficiently favorable result, the putting down of a trial well was begun. This well was lined with cast-iron slotted cylinders, 16 ft. in diameter, double dipped in asphalt. The lowest cylinder had a thick cutting edge, and was followed by three of the slotted cylinders, each 3 ft. 4 ins. high. Upon the topmost was placed a flat ring, and on this the masonry lining of the well was built up to the surface in a water-tight manner.

This well was put down to a depth of 43 ft. beneath the surface of the ground, by means of dredging under the cutting edge. After its completion water was pumped from it to the amount of about 1 250 000 galls. (5 000 cu. m.) daily for several weeks, day and night, during which time the well yielded, on demand, by means of an increased lowering of the water level, up to 2 000 000 galls. in 24 hours. The level of the water-table was determined during the regular pumping of 1 250 000 galls. in 24 hours by means of measurements in bore holes put down in lines, each parallel to the other, and at right angles to the river. In determining upon a proper depression of the level of the water in the well, the rule was observed not to allow the water-table to intersect the level of the water of the river itself.

During these examinations, it was shown that the subsoil was very solidly packed, and the water-table, when the water in the well was lowered from 20 to 23 ft., and only drawing 1 250 000 to 1 500 000 galls. in 24 hours, would slope sharply toward the well, and would gradually curve over, so as to join the original water-table at a distance of 260 to 300 ft. from the well.

An examination of the quality of the water taken from the bore holes and from the trial well was made in two laboratories, in the Government Chemical Central Laboratory at Dresden, and in the chemical laboratory of Dr. E. Kayser, of Dresden.

The table on page 34 will show the result of such chemical analyses.

According to these analyses, the ground-water thus discovered is of exceptional purity, due to the very small quantity of organic substances contained and the absence of every trace of nitrous acid and of ammonia. The amounts of all the other substances contained are less than the limits which have been fixed for the determination of a supply of pure and wholesome water, and it may be assumed, as indicated by a comparison of the analyses of the samples taken on November 7th and 12th, that these quantities will be still further reduced by continued pumping, and by covering over the well, because they are derived partly from the lining of the well, and partly from the unavoidable accident of substances getting into the well from above.

The maximum amount of ground-water could be drawn on the site selected, by the construction of a collecting gallery, similar to the one put down at the present water works, at a depth of 16 to 20 ft. But it costs a disproportionately great sum to build such a gallery at a depth of 40 to 43 ft., as would have to be done on this site. The plans, therefore, are to put down a row of deep wells, somewhat after the manner in which the trial well was put down, to string them along the site in question, and connect them with the new pumping station. This station will be placed on the site between the Blasewitz-Tolkewitz and the Dresden-Tolkewitz Street, and above the high water-line of 1845. It is for the present to consist of two steam pumps, each of 5 000 000 galls. capacity in 24 hours, provision being made for placing two other steam pumps of this capacity. A force main of 28 ins. diameter (interior) will be laid through the eastern part of the old city to the reservoir to be constructed on Räcknitzer Hill, holding about 3 000 000 galls. It is intended to lay a second force main of the same diameter and build another reservoir of the same capacity at a later day. The distribution system of a portion of the old city will be connected with these force mains.

The high-water level of the new reservoir is nearly the same as the high-water level of the reservoir at Fish-House, on the right bank of the Elbe, which has a capacity of 5 000 000 galls.

Water is to be pumped directly into the old city from the new water works through the force mains, and to connect with the old city distribution pipes.

A From Bore Hole No. 18.				B From the Trial Wells.				From Present Water Works.
Date when Samples were Taken in 1892.								
August 12.	November 7.	November 7.		November 7.	November 21.	November 7.	November 21.	August 12, 1891.
Analyses by—								
Dr. Kayser.	Dr. Kayser.	Government Chemical Central Laboratory.		Dr. Kayser.	Dr. Kayser.	Government Chemical Central Laboratory.	Government Chemical Central Laboratory.	Government Chemical Central Laboratory.
One liter of water (1 000 gr.) contains :								
Gram. 0.2244	Gram. 0.2014	Gram. 0.1902	Solid matter in solution.....	Gram. 0.2260	Gram. 0.2010	Gram. 0.2074	Gram. 0.2028	Gram. 0.1569
Of which—								
0.0012	0.0015	0.0001	Organic substances.....	0.0035	0.0011	0.0004	0.0001	0.0020
0.0230*	0.0204	0.0141	Chlorine.....	0.0213	0.0195	0.0153	0.0145	0.0113
0.0433	0.0336	0.0278	Nitric acid.....	0.0362	0.0282	0.0297	0.0297	0.0096
0.0485	0.0490	0.0450	Chalk	0.0524	0.0513	0.0510	0.0474	0.0422
0.0113	0.0129	0.0113	Magnesia.....	0.0142	0.0149	0.0125	0.0123	0.0124
	0.0276	0.0275	Sulphuric acid.....	0.0358	0 0345	0.0356	0.0334	
	0.0140	0.0187	Salicic acid.....	0.0146	0.0152	0.0181	0.0190	
	0.0154	0.0220	Fixed carbonic acid.....	0.0163	0.0149	0.0216	0.0202	
			Potassium	.....	.....	.....	0.0058	
			Soda.....	.....	.....	.....	0.0266	
			No ammonia					
			No nitrous acid.....					
			No phosphoric acid.....					
6.7	6.7	6.08	German degree of hardness.....	7.2	7.1	6.85	6.46	5.96

A From Bore Hole No. 18.				B From the Trial Wells.				From Present Water Works.
Date when Samples were Taken in 1892.								
August 12.	November 7.	November 7.		November 7.	November 21.	November 7.	November 21.	August 12, 1891.
Analyses by—								
Dr. Kayser.	Dr. Kayser.	Government Chemical Central Laboratory.		Dr. Kayser.	Dr. Kayser.	Government Chemical Central Laboratory.	Government Chemical Central Laboratory.	Government Chemical Central Laboratory.
From the figures above given, the amount of substances dissolved in 1 liter of water are as follows :								
	0.0469	0.0468	Sulphate of lime.....	0 0609	0.0586	0.0605	0.0568	
	0.0350	0.0459	Carbonate of lime.....	0.0371	0.0254	0.0466	0.0428	
	0.0215	.....	Silicate of lime.....	0.0229	0.0248			
	0.0029	0.0187	Free silicic acid.....	0.0028	0.0024	0.0181	0 0046	
	0.0460	0.0381	Nitrate of magnesia.....	0.0496	0.0386	0.0407	0.0407	
		0.0021	Carbonate of magnesia.....	.....	0.0072	0.0021	0.0026	
	0.0020	.....	Chloride of magnesia.....	0.0027	0.0043			
	0.0312	0.0232	Chloride of sodium.....	0.0318	0.0268	0.0252	0.0239	
	0.0015	0.0001	Organic substances.....	0.0035	0.0011	0.0004	0.0001	
			Silicate of soda.....	.....	.....	.....	0.0218	
			Silicate of potash.....	.....	.....	.....	0.0095	
	0.0144	0.0153	Crystal water.....	0.0147	0.0118	0.0138		
	0.2014	0.1902		0.2260	0.2010		0.2028	

* Common salt 0.0380 gr.

The new pumping station is designed for a maximum consumption of 15 000 000 galls. in 24 hours, so that the two water works of the city of Dresden will have a joint capacity of 27 500 000 galls. in 24 hours.

The experience of the year 1892 has shown that great caution should be exercised in determining upon the actual amount of consumption. The long-continued drought of that year caused an excessively large use of water, and but few water works were in a position to meet the same. It was found that the maximum consumption in that year in Dresden, was over 40 galls. per day per head, and the water-works had to restrict the amount supplied to the public fountains, so as to provide for the event that the consumption would increase still further.

Inasmuch as the time of maximum consumption coincides very nearly with the period of the lowest yield of ground-water, it is necessary, in designing works of this kind, to compare the maximum consumption with the minimum yield, instead of with average quantities, as is often done. The copious yield of some water works, at times of the year when there is only a small consumption, has no value, because it is impracticable to store water to the extent of utilizing it during the warm months of the year, as has been shown by experience.

In the case of many water works, statistics have shown that the maximum consumption per head of the inhabitants supplied by the works, has been, on many days, considerably greater than the above-named measure of 40 galls. per day, and it will be necessary in computing the available quantity to provide for larger figures.

The above-cited examples are sufficient, without mentioning the great number of smaller works for smaller cities, to show to what extent the use of ground-water has established itself in the last 25 years, and it may only be mentioned in conclusion that the author has been called, in conjunction with Mr. O. Smreker, Civil Engineer, of Mannheim, to make plans for furnishing the city of Vienna with ground-water, in addition to its high spring aqueduct.

These works are to be of a capacity of 50 000 galls. per day, and are to be designed so that they may be increased to a capacity of 100 000 000 galls. Mr. von Wessely, of Prague, principal of the firm of Corte & Co., contractors, of Prague, has been charged with the conduct of the preliminary borings.

Bibliography.

"Water Works of the City of Halle." By *B. Salbach.* G. Knapp, Halle, Publisher, 1871 (now E. Nowak, Publisher, Leipzig).

"Water Works of the City of Dresden." By *B. Salbach.* G. Knapp, Halle, Publisher, 1874 (now E. Nowak, Publisher, Leipzig).

"Water Works of the City of Crefeld." By *B. Salbach. Glaser's Annalen,* Vol. VIII, Part 103 (of the whole series 85 and 87), 1881.

"Water Works of the City of Groningen." By *B. Salbach. Glaser's Annalen,* Vol. XI, Part 4, 1882. (No. 124 of the whole series.)

"Water Supply of the Upper Silesian Factory District." By *B. Salbach. Glaser's Annalen,* Vol. X, Parts 1 and 2, Nos. 109 and 110.

"Water Supply of the City of Cologne." By *E. Genzmer. Journal für Gasbeleuchtung und Wasserversorgung,* 1889. Page 89 to 95 and 123 to 131.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

648.

(Vol. XXX.—November, 1893.)

SOME QUESTIONS CONCERNING THE FILTRATION OF WATER.

By W. KÜMMEL,* C. E., Engineer of Gas and Water Works Company at Altona, Germany.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

The Imperial Board of Health in Berlin was led by the violent epidemic of cholera in Hamburg to compile, some months ago, the principles derived from the experience of water works with sand filtration, in order to avoid infection when cholera is prevalent.

These principles, translated in somewhat abridged form, run thus :

First.—Care should be taken that the rivers, ponds, etc., from which the water is derived be safely protected against pollution by human dejections ; no vessel, barge, etc., should be allowed to anchor near the intake of the water works.

Second.—Sand filters do not deliver the water totally free from microbes ; they restrain only a certain number of them, including the

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

* Died at Chicago, July 19th, 1893

microbes of the cholera; therefore the filters should never be overworked.

Third.—The velocity of filtration ought not to exceed 4 ins. (100 mm.) per hour.

Fourth.—If the quantity of water consumed in the town surpasses the quantity that can be delivered by the existing filters at the said velocity, the water works have to remedy the defect, either by reducing the consumption (introduction of water meters) or by building new filters.

Fifth.—The stratum of the filtering sand should never be less than 12 ins. (30 cm.) thick.

Sixth.—When a filter has been cleaned, or filled up with new sand, it is necessary to waste the first filtered water, with its high number of germs, and not deliver it to customers.

Seventh.—The effect of filtration must be daily controlled by bacteriological tests of each filter. If the bacteriologist finds suddenly a greater number of germs, or unusual species of microbes, in the filtered water, such water should not be used at all.

Eighth.—If these principles are carefully followed, the danger of the microbes of cholera passing through the filtering sand-stratum is exceedingly small, as has recently been proved by the Altona water works compared with Hamburg.

The author of this publication is believed to be the celebrated Doctor Robert Koch, at Berlin, the discoverer of the microbes of phthisis and of cholera, who has for many years been making inquiries into the effect of filtration, and these rules are, therefore, the result of a large experience. The writer is sure that the majority of his colleagues who are busy with filtration will, in conformity with their own experience, accept most of them; but some will be in doubt whether all the rules are exactly correct.

The publication of the Board, really a governmental order in Germany, settles the maximum velocity of filtration not to exceed 100 mm. per hour, or 2.40 m. per diem, equal to a delivery of filtered water per square meter per diem of 2.40 cu. m.; that is, 4 ins. per hour or 8 ft. per diem velocity, and 60 U. S. galls. (50 imper. galls.) delivery per square foot per diem in English measures.

As far as the writer is instructed about the rate of filtration now used in English water works, he finds these numbers corresponding

with those in use there, as an average in some works, but as a maximum in others. It is not very many years ago that in England the average quantity of filtration was about 50% more than 8 cu. ft. per square foot of the filtering area ; and this quantity has been delivered in Germany, too, chiefly by water works having a water with few earthy ingredients. Why shall we take the 4-in. velocity as a standard for a safe and sure filtration ?

Mr. Bertschinger, chemist and bacteriologist of the corporation of Zürich, published a paper in 1889 concerning "The Working of Sand Filters in Zürich." In this paper he states, from a long series of trials, "that the result of the filtration is equal to velocities of from 2.7 to 13.4 m. (or from 9 to 44 ft.) per diem."

Mr. Piefke, engineer of the Stralau station, Berlin water works, writes in 1887* that his filter delivers the water nearly sterile at a velocity of 30 mm. per hour, or 30 ins. per diem; that the velocity of 60 to 80 mm. per hour, or 4 ft. 9 ins. to 6 ft. 4 ins. per diem, would be for his works tolerable; but 100 mm. per hour, or 8 ft. per diem, should be the maximum for new works. Notwithstanding, it is stated in another paper, issued by Dr. C. Fränkel and Mr. Piefke, that on March 12th, 1889, the Stralau water works filtered with a velocity of 224 mm. per hour, or 17 ft. 11½ ins. per diem. The bad working of the filters has caused the authors named to make a series of trials with pathogenic and non-pathogenic microbes, from which they have stated that the pathogenic microbes pass the filter even at a very low velocity of filtration ; but—it may be added—under the supposition that the raw, unfiltered water be mixed with the microbes at the rate of 500 to 700 per cubic centimeter, and the filter be ill-treated as it was by the authors of these tests. Messrs. Fränkel and Piefke conclude from their experiment that filters should not be worked with a velocity exceeding 50 mm. per hour, or 4 ft. per diem.

Mr. Piefke two years later repeated the experiments, without repeating the mistakes of the first series. The result was nearly the same ; but he worked again with a water containing a number of pathogenic microbes, such as we can absolutely not find in a polluted river or pond used for water delivery.

The writer himself has also made a great number of trials to find the best velocity for the filtration of our water. To avoid every error

* Principien der Reinwasser-Gewinnung mittelst Filtration.

or mistake resulting from the difference in the quality of the unfiltered water, he built three experimental filters, filled with the same filtering materials, all conditions being exactly the same, only the velocity of filtration being different. As soon as the three filters were in good condition the experiment began, the water running through the filters at a velocity of 50, 100 and 200 mm. per hour, or 4, 8 and 16 ft. per diem. The result was that the number of microbes in the filtered water was, without exception, the highest in the water of 50 mm.; averaging, higher in the water of 100 compared with that of 200 mm., but really there was no great difference between the three. The number of the microbes varies in the 50-mm. water from 11 to 97; in the 100-mm. water, from 5 to 79; in the 200-mm. water, from 7 to 72, per cubic centimeter. The difference is not sufficient to declare the one velocity better than the other, and therefore the writer cannot agree that the velocity of 100 mm. is, beyond doubt, the maximum of safe filtration. Notwithstanding, the writer accepts this standard as long as better experiments do not prove it to be false, because he believes the danger of a trespassing pathogenic microbe is much more trifling at the lower than at the higher velocity; and, besides, that the best velocity is not the same for all different qualities of water. He is sure that the difference in the mineral, vegetable and animal admixtures is of high importance in this question, and that we should endeavor to find out the best velocity for each separate water and water works.

A second point that is often disputed is contained in Clause 6. According to the present state of science and experience, it is generally accepted that the thin film of dirt on the surface of the sand is really the filtering medium, and that the filters, filled up with new sand or cleaned, do not properly work before a new film of dirt has been gained. Mr. Bertschinger's experiments show that the Zürich filters do not do their full duty until four to seven days after the cleaning. Mr. Piefke's experiments at Berlin show the filter fully restored after four to five days' working. The unfiltered water in Zürich is of a first-rate purity compared with the water of the Spree in Berlin and of the Elbe near Altona. The raw Zürich water contains generally less than 300, in very rare cases 400, germs, the water of the Spree generally some thousands, very seldom more than 10 000 germs, while the water of the Elbe at the Altona water works contains exceedingly seldom less than 10 000, generally more

than 20 000, and very often more than 40 000, germs per cubic centimeter. On the other hand in the Elbe water the amount of mineral substances is very high, the amount of vegetative matter very low, compared with the Spree water. The mineral matter subsides very fast, as soon as the water is brought to rest. The writer, therefore fills up the cleaned or newly filled filter in the usual way from below with filtered water to the top of the sand, and afterwards with unfiltered, subsided water from above, and then gives a sufficient time for the mineral impurities to subside on the surface of the sand. Generally six to eight hours, sometimes 10 or 12 hours after the time the filter is filled with water, the filtration begins and is quite satisfactory, the number of microbes being not, or very little, increased, as compared with the water filtered before the cleaning.

Quite a different case is that after the filling up with new sand. If we proceed just in the same way with the inlet of water, and give the raw water a time of 20 hours of quiet rest, the number of microbes becomes very high, as the following figures may show :

	Number of microbes.	
Before the filling up	42	24
One day after filling up	1 880	2 300
Two days after filling up	752	1 364
Three days after filling up	208	468
Four days after filling up	156	244
Five days after filling up	102	82
Six days after filling up	84	116

It is certain that the working of the filter, when filled up with new sand, is for a certain time not satisfactory ; therefore, we must waste the water during the first three or four days unused.

A third point to be discussed is contained in Clause 7 ; it is not sufficient to examine the water as it is delivered to customers ; the water running from each filter must be every day tested bacteriologically. We have made arrangements, to take the samples from each filter, as near as we could get. At first there were attached to the outlet pipes, short lead pipes with bib cocks, and the water was drawn from these cocks, about 2 ft. from the mains. That was a mistake ; some days after the first trial the lead pipes were grown full with microbes, and the samples of water taken from the bibs, though full care was taken to have the water from the mains, did not show the

latter, because the running water washed away many of the colonies in the lead pipes. Now the arrangement is altered; we take the water directly from the main with a sterilized short pipe, introduced in the main through a properly constructed stop-cock.

We find it a very good thing to examine every day each of the filters, and we are sure that we save ourselves a great many mistakes and much mischief by this very simple and very safe expedient. Without this bacteriological examination we are working just like a mechanical engineer who does not indicate his steam-engine, thinking that he is able to estimate its quality from the revolution of the fly-wheel.

The only thing we have not to overlook is that the number of germs found in a cubic centimeter is not very important in itself, but only a means and way to find out the good working of a filter. As soon as we find that the number increases suddenly, we may be sure that something in our filter is out of order, and we have to do our best to search and repair the fault.

A point of much greater importance is the determination of the species of the microbes ; in each sample of a good filtered water the number of species will not exceed five or seven, and these species will not vary very much, though different species predominate at different seasons. As soon as we find that the water contains some well-known species of microbes of the liquefying type, we have to examine the colonies in the discs and the filters with the utmost care.

If we do so, we may be sure that we are able to deliver a good safely usable filtered water, even from a river not at all protected against pollution through human dejections.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

649.

(Vol. XXX.—November, 1893.)

ON THE DISTRIBUTION OF WATER IN THE CITY OF MEXICO.

By L. SALAZAR, M. Association of Engineers and Architects of Mexico.

Translation.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

Sources of Supply.—Tradition has fixed the year 1465 as the epoch in which the works were completed for bringing to the city of Mexico the water from the springs of Chepultepec, declaring that the enterprise was fathered by Metzahualcoyotll, who, it is said, did the leveling.

Two springs, called, respectively, Alberca Chica and Alberca Grande, are certified as actually belonging to the municipal dominion. The water which they produce is denominated "heavy," in distinction from the water of the Desert, called "light," from its lighter density. The

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

"heavy" water of Chepultepec is transparent, of a bluish color, and has an average temperature of 21.5° Cent. Its density is 1.00084 referred to that of pure water at 0.76 m. above sea level and 4° Cent. According to the learned Mexican chemist, D. Leopold Rio de la Loza, the analysis of the heavy water gave him the following result:

Gaseous products:

Air	10 390
Oxygen	1 760
Carbonic acid	0 990
Total in cubic centimeters per liter	13 140

Solid products in grams per liter:

Sulphate of lime	0.00652
Carbonate of lime	0.02712
Carbonate of magnesia	0.02215
Carbonate of soda	0.03901
Chloride of soda	0.05845
Silicate of soda	0.02997
Acetate of potash	0.02158
Silica	0.07745
Alumina and iron	0.00686
Organic matter	trace.
Lost	0.00093
Total in grams per liter	0.29004

Temperature in the springs, 22.5° Cent.; density, 1.00028

The two Albercas contain great numbers of small fish, called by the Aztecs, "Fentzonmichi," whose existence and facile development in the springs is one of the most important biological features of this potable water.

The water of Chepultepec was considered insufficient for the necessities of the population from the first years following the conquest of Mexico. It was on this account that, in 1576, the water from the springs of Santa Fé was introduced into the city, first, by a channel

in masonry, and afterwards, in the ditch or aqueduct of San Cosme, which was completed in 1620.

The works for carrying this water were of considerable importance on account of the broken nature of the ground, where they are developed. The conducting channel is notable for the epoch in which it was built; the water-conducting bridge, 30 m. high and 37 m. long, which was finished in 1734, and the grand arcade, composed of more than 900 arches of stone masonry and light brickwork, having each a span of 6.70 m., with 55 m. of height and 1.46 m. of thickness, the canal being 0.63 m. deep and of equal width. At the present time this aqueduct has disappeared within the western limits of the city.

The Santa Fé spring is 10 km. from the city of Mexico and 166 m. above the level of the city. The temperature of the water in the spring is 15° Cent.

From the end of the last century, in 1786, there have been added to the waters of Santa Fé, those of the springs of the Desert and Los Leones brought from their source in the western wooded district of the valley, about 25 km. distant from the city. All this water is designated as "light," to distinguish it, as already explained, from that of Chepultepec, which has been called "heavy." The waters on leaving the springs are diaphanous, but, being stirred up in their passage through the aqueduct, become turbid and require filtering before use, especially in the rainy season, since at some points the canal, not being covered, the dust in the atmosphere reaches it, and also earth loosened by rains at the head.

The springs of Los Leones have their origin in the mountain of San Miguel. There are many little springs disseminated in all directions, which are caught in their natural channels by the dam of Los Leones, where the works of masonry begin, and where also the waters of the Desert are gathered.

The distinguished Mexican engineer, Señor Don José Salazar Illaregui, who was the chief of the Boundary Commission with the United States of America, did the leveling and designed the plans of the work from the city to the dam of Los Leones, the starting point of the work being the first arch of the aqueduct in the Avenue of La Veronica, the plane of comparison being the bottom of the superior canal. The results were as follows:

	Distances. Meters.	Heights. Meters.
Arch of La Veronica (Initial Station)..		
Aqueduct Union	1 408	2.09
Mill of Belem	6 980	128.35
Springs of Santa Fé	9 716	166.18
Angostura	11 970	340.79
Tres Cruces	19 582	672.71
Junction of the canals	19 732	680.80
Head of the Desert aqueduct	24 932	859.49
Head of Los Leones aqueduct	24 580	904.21

The springs in the mountain of San Miguel are still higher than the last mentioned by 27 m.

The analysis of these waters by Señor Leopold Rio de la Lozo, already mentioned in treating of the Chepultepec Springs, gave the following results, viz.:

Temperature at the sources being 9° Cent.; density, 1.000267.

Gaseous products, in centimeters per liter:

Air	10.151
Oxygen	2.809
Carbonic acid	0.750
Total	13.710

Solid products, in grams per liter:

Sulphate of lime	0.00326
Carbonate of lime	0.02171
Carbonate of magnesia	0.01169
Chloride of potassium	0.00396
Chloride of magnesia	0.00349
Silicate of soda	0.03985
Silica	0.05169
Alumina and iron	0.00849
Organic matter	0.00087
Total	0.14501

The northern district of the city, known as the Santiago Tlaltelolco Ward, was provided with water from the springs of Motlatica or Xancopinca. In 1618 these waters had been already mentioned by a historian of the epoch, and at the end of that century were still used in the supply of the city, according to the Franciscan chronicler, Betantourt. Whether it be that the pavement of the zone supplied by the water of Xancopinca has sunk, or that the spring has lost (in some other way) its ascensional force, the fact remains that this water no longer arrives at Tlaltelolco, this being one of the causes of the decadence of that ward.

In 1847 Dr. Leopold Rio de la Loza examined said water and found it superior to the heavy water, since its density was 1.000201 and it contained only 0.15011 of fixed substances in 1 000 parts of the liquid. The color of the water and its transparency are similar to the others of which mention has already been made. The temperature in the spring is 21.50° Cent.

The spring, on account of the duration of time in which it had been abandoned, reverted to the possession of private owners, being claimed by the abuttors. The Municipal Council of 1892 succeeded in re-establishing the rights of the city and at the present time are occupied in constructing the works necessary to restore the waters to the district which formerly enjoyed their use.

The increase of population in the wards known as Santa Maria Guerrero, Los Angeles, Santiago, Tlaltelolco and Santa Anna, and the almost absolute want of water in the district, induced the Municipal Council of 1882 to acquire by lease a part of the water that supplied the city of Guadalupe Hidalgo. That water proceeds from Tlalnepantla, 13 km. distant from Guadelupe, and its aqueduct was constructed in 1743–51 at a cost of $129 000, collected by popular subscription.

From the city of Guadalupe to Mexico the pipe line was laid in the short space of 95 days, and for the first time in our history the material was of Mexican manufacture, giving advantages both in its superior quality and in its cheaper cost.

The introduction of this water into the city of Mexico was inaugurated on April 2d, 1882, realizing thus an important improvement for one of our most populous neighborhoods, which has had a notable advance in population since that event.

It could not fail that the experiment of artesian wells should be tried in Mexico, and it has fallen to Messrs. Pane & Molteni to be the first to have achieved success in such work, which has given them the best results. The persistency and labor of the gentlemen conquered all the difficulties of an enterprise entirely experimental, and in May, 1857, they could count 168 of these wells opened, of which 24 were intended for irrigation, 120 belong to private houses, while 24 sunk by other contractors produce conjointly 690 cu. m., 639 liters per hour.

According to the data collected by the Government in 1883, there existed at that time 483 artesian wells, distributed in the city as follows:

1st Ward	24	5th Ward	40
2d "	55	6th "	39
3d "	61	7th "	47
4th "	40	8th "	177

The advance made in the opening of artesian wells shows the favorable conditions of the ground for the perforation of this class of works. The waters from these subterranean springs are as good as the potable waters already described, inasmuch as they proceed from springs situated outside the influence of the infected waters near the surface. The augur penetrated 234 m. in the well opened in the Military Hospital, and depths of 150 m. are by no means rare. The well sunk in the Mint in 1871 reached a depth of 149.162 m., and is one of the best exposures for the study of the geology of the region. Thirty-nine different strata are passed in which the permeable alternate with the impermeable, while the lacustrian condition of the deposits remains the same as in the running streams. In this well are three principal gushing springs at the depth, respectively, of 62, 104 and 141 m. The strata containing springs are in sand and overlie the argillaceous.

Water from the well of Bucareli was analyzed by Dr. Leopold Rio de la Loza, with the following results:

Gaseous products:

Air and oxygen	13.15	Carbonic acid	1.18

Being a total of 14.33 cu. cm. per liter.

Solid products in grams per liter:

Carbonate of lime.....	0.01130	Silicate of potash.....	0.01040
Carbonate of soda.....	0.06949	Silica.....	0.04515
Chloride of potassium.	0.00086	Alumina and iron.....	0.00276
Chloride of magnesia..	0.00091	Organic matter.....	traces.
Silicate of soda.....	0.08375		

Making a total in grams per liter..... 0.12462

The average temperature of water in the wells has been found at 15.5° Cent.

The number of artesian wells actually in use in the city of Mexico to-day is 331, which, however, do not contribute all they ought to the general supply, since their product is applied exclusively to the use of the property on which they happen to be placed, leaving the surplus flow to be wasted.

Quantities of Water.—The works that have been published concerning the water supply of the city tell us what is the character of the spring and spouting well water, which has been given the population. Only in 1857 Mr. Pane gave the following figures indicating the volumes of water in the various sources received by Mexico in cubic meters:

	Cubic meters.	Liters.
Superior aqueduct in La Cosme, light water.....	586	718
Inferior aqueduct, heavy water.....	287	117
Aqueduct of Belem, heavy water	363	058
Artesian wells.....	690	639
Total per hour.....	1 927	532

In 1864 Señor Don Manuel Orozso y Berra calculated the demand of the capital for water at 250 liters per day per individual in a population of 200 000. His figures were:

	Cubic meters.	Liters.
Waters, heavy and light.....	1 233	893
Waters of the artesian wells.....	866	895
Total used in the city per hour	2 100	788

In 1883 the data collected by the Government of the Federal District produced results as follows :

Light water	11 520	cu. m.	in 24 hours.
Heavy water	6 048	"	" "
Guadalupe water	1 152	"	300 liters, in 24 hours.
Total supply in 24 hours	18 720		300 "

The water produced by the artesian wells does not enter into this account because so much of it is lost in waste and none of it is used for the general public.

Considering that the population of Mexico was 300 000 in 1883, this amount of water would give 64.4 liters to each inhabitant ; without reckoning that the volume of water so received has served, not only for domestic uses, but for the public service and for industries.

At the present time the water supply of the city has experienced notable modifications on account of the works executed for introducing it under pressure, a matter to be hereafter treated. This quantity is to-day as follows :

	Cubic meters.
Heavy water from the two Albercas of Chepultepec	25 920
Light water from the Desert, Los Leones and Santa Fé	10 080
Light water from Guadalupe Hidalgo	1 440
Gushing waters from artesian wells	7 690
Total supply in 24 hours	45 130

According to the census of 1890, the present population of the city of Mexico is 326 394. The quantity received as above noted does much more duty than supply this population with what it requires for its domestic use ; it is devoted to the irrigation of the streets, plazas and public gardens ; to extinguishing fires, washing houses, carriages and stables; also for animals used for work purposes, and in the industrial arts. Nor is it possible to estimate the quantity devoted to each kind of service, although it is well understood that there is a positive deficiency for general domestic use. This need has been emphasized in the present year by the increased pressure in pipes, which, being still a novelty, has not yet met the proper regulation in private houses.

The efforts of the Municipal Government are now directed to securing other sources of supply and perhaps the establishment of a double service, one for domestic uses exclusively, and one for the other uses above designated, as is the custom in the cities of Europe and the great cities of the United States of America.

Water-Courses and Conduits.—The light waters proceeding from the Desert, Los Leones and Santa Fé are conducted to the fall of the Molino del Rey in earthen pipes, following the topographical contours of the ground, except that, where the depressions are severe, aqueducts in masonry have been constructed, as already stated. Along the 25 km. of pipe line, as far as the Gate of the Tlaxpane, are the pueblos of Cuajimalpa, Santa Fé, Tacubaya, which use the water, as also the flouring mills of Santa Fé, Valdez, Santo Domingo and El Rey, the paper mills of Belem, the pin factories and the National Artillery Foundry at Chepultepec.

From the Molino del Rey fall, which the city acquired in 1891, the water continues its course in a pipe of 0.80 m. diameter, parallel to the old aqueduct from the woods of Chepultepec, and from La Veronica passing by Tlaxpana to the crossing of the front of the church of San Cosme. At this crossing the pipe line trifurcates and begins the distribution of the thin water supply for two-thirds of the city's area.

The heavy water produced by the springs of Chepultepec is raised by mechanical means to elevated reservoirs, hereinafter described, and from these is distributed according to the lines of the plan, which run parallel to the old aqueduct of Belem. At the Bucareli crossing begins the distribution of water for the remaining third part of the municipal area. The light water proceeding from Tlalnepantla, and acquired from the City Council of Guadalupe Hidalgo, comes in a pipe line from that town, following the wagon road, which starts directly from Pehalvillo. As already described this water supplies the district of the north of the capital, where it was difficult to take other light water under pressure before the construction of works lately executed.

The distribution of water in the city of Mexico presented great and grave defects until within a very few years. The pipes were of lead, of various diameters, recognizing no law, and were, moreover, deteriorated, principally by the water which leaked from them. The Municipal Council, recognizing the necessity of doing something to improve their very bad condition, and being without the funds required

for the undertaking, in 1880 called for a loan of $500 000 for the prosecution of this important work.

The project, which had been in the hands of the Council since 1868, had been made by the engineer, Don Casimiro Pacheco, and approved by the Commission appointed for the purpose, and also by the Water Commissioners of the Municipal Corporation. But the 12 years transpiring to 1880 had created new requirements, which demanded a change in the solution of the problem. There was, moreover, as a limitation to the work, the products of the loan already mentioned. Fortunately, the Engineer Director of Water, Señor Manual Patiño, had for 10 years employed himself in gathering data of a practical character, which now enabled him to present a plan commensurate with the requirements of the problem.

The Water Commission of the Municipal Council, presided over by the distinguished engineer, Señor Leandro Fernandez, made a minute study of the above-mentioned data, and formed a project, which was approved by the Council. As a base of the project, it was estimated that a supply could be counted on of 8 000 liters of light water, and 5 000 of heavy water, per minute, in accordance with which the city was divided into districts, the caliber of the pipes being arranged with reference to the slight declivities of the streets and the density of population in any district. The project was nearly abandoned on account of the failure of success with the loan of $500 000, but the efforts of General Carlos Pacheto, then Governor of the Federal District, opened the road for the enterprise. At his suggestion some of the municipal property was sold and the proceeds availed of to contract in England for 2 500 tons of pipe and accessories for the principal branches of the distribution. This work commenced in April, 1882, and continued without interruption under a plan more comprehensive than the project of 1880, the annual appropriations being as follows:

1881	$89 523	1887	$12 650
1882	185 456	1888	20 000
1883	179 662	1889	136 118
1884	56 485	1890	189 278
1885	11 752	1891	261 100
1886	34 864	1892	39 000
The total amounting to			$1 245 888

Meanwhile the disbursements were as follows:

1881	$51 302	1887	$41 486
1882	33 560	1888	48 706
1883	30 069	1889	61 140
1884	30 899	1890	117 905
1885	35 490	1891	177 732
1886	43 156	1892	134 347

The total of these disbursements were thus..... $805 792

The length of pipe line for the distribution of the various classes of water exceeds 100 km., the diameters of the varying sizes being for the principal lines 80, 60, 50, 40, 30 and 25 cm., respectively, while for the lesser lines they are 14, 12, 10 and 8 cm.

Also, there were placed at all street crossings the requisite gates, made in the machine shops of Mexico for facilitating repairs, without interfering with the general distribution. The same shops made the bronze faucets in use, and prepared the dead ends of pipes with lead at the entrance of the houses, for joining up the distribution within the domicil. The water is led by a principal pipe to the cistern, in which is arranged a safety cock (*llave economica*) with a float, to prevent the prodigal waste of the fluid. From this cistern the occupants of the house can make freely such distribution as suits them, with the sole proviso that they put a cock at the end of every branch in the house.

The operation of pumping is practiced only for the water of Chepultepec. That proceeding from the Desert, Los Leones and Santa Fé is found under conditions which permit its introduction to the city by gravity, running in pipes from El Molino del Rey, situated at an elevation of 26 m. above the average level of the city. The heavy water of the two springs of Chepultepec is collected in a tank beside the engine-house containing the pumps, by means of a pipe laid at the bottom of the springs, of 80 cm. diameter, whence, through pipes of 60 cm. diameter, it is raised to another reservoir 25 m. above the edge of the hill of Chepultepec, by three English-built Worthington pumps, with a capacity of 1 020 cu. m., each motor being of 70 H. P. These engines are of the kind known as "high pressure, direct acting, ex-

pansion, with condensers," each one possessing the ability to raise 12 cu. m. of water per minute under a head of 25 m.

The cylinders have the following dimensions: The high-pressure cylinder has 8 English ins. diameter, the intermediate, 13, and the low pressure, 21; all have the same length of stroke, viz., 18 ins. The pumps are of 17 ins. diameter.

Working under normal conditions, the engines make 45 revolutions per minute. The reducing valves are moved by levers connected with the piston valves by means of eccentrics (*engranes*). The valves are so arranged that the cylinders take steam throughout the stroke, the expansion occurring only by the passage of the steam from one cylinder to another, thus securing the greatest simplicity of construction and avoiding difficulty of adjustment. The valves on one side of the engine are moved by the trip of the opposite piston, and its combination permits the constant uniform flow of water. The condenser is of the type called *chorro*, and the pump for it is connected with the piston rods, as are also the principal pumps, which are double acting, and the valves of which are provided with springs. Each engine weighs 20 tons. The normal pressure of steam in the boilers is 100 lbs. per square inch. The normal velocity of the engines is 135 ins. per minute, which corresponds to the revolutions already noted. The motors consume 2 lbs. of combustible* per hour per horse-power developed. The combustible is brought from the woods of Desierto, property of the Municipality of Mexico, the exploitation of which is in charge of an agricultural professor (*agronomo*).

The building containing this machinery is in three divisions; the pump-room measures 35 ft. by 32; the boiler-room, 33 by 32; and the fuel-room, 32 by 32. The iron chimneys are 50 ft. high above the brick foundations on which they rest. The three pumps with their motors, boilers and erection, including the building in which they are located, cost $100 000, and were inaugurated December 29th, 1891.

Uses of the Water.—The potable water made use of by the city of Mexico is variously employed. It is understood that for drinking and domestic purposes it is necessary to employ salubrious water since hygiene demands that such water shall not contain germs prejudicial

* Nothing in the paper indicates the character of the "combustible," which is certainly not mineral coal, as it does not exist in the region. It is probably wood, though charcoal is a common fuel in the city of Mexico, and its transportation would be much less expensive than wood.—*Trans.*

to health, whether it be solely for drinking, for baths, for daily toilet use, or for washing clothes and other articles of domestic use. But for irrigation, for extinguishing fires, and for certain industries, it is a damage to use potable water unless the service can be regulated, since it often prevents the preferable utilization of the article for private health. In the city of Mexico these inconveniences are especially felt in the dry season, immediately preceding the rains, when the springs are reduced in their natural supply. Happily this trouble will last but a little longer, since the municipality will acquire in a very short time new supplies which will be conveniently distributed for private domestic use, in a system under a head of 25 m., not permitting the present enormous waste.

Water is delivered to private parties for the supply of their establishments through the medium of a rent contract, by which a monthly payment of $2.50 gives a *medio merced** of water, equivalent to 1.15 liter per minute, and which is the minimum volume prescribed for each house. When the water is delivered under a 25-m. head, the price will be advanced to $4. The supply of water may be augmented in any case in units of *medias mercedes* at the same proportional rents. At the present time there are 2 543 houses which employ and pay rent for light water, 1 860 which use the heavy water, and 116 which use the water from Guadalupe Hidalgo, being a total of 4 519 houses.

By some old concessions there are certain establishments which may use potable waters without payment of rents. There are 239 houses presenting such claims, but up to the present time the municipality has recognized only 175 of them. Free use of water is also conceded to the municipality and to those who have artesian wells in their houses, amounting in number to 234, as registered. Public edifices enjoy concessions for the free use of water, and also some private houses occupied by the Government offices or the municipality, while they are so employed. Public fountains for the free use of the people have been superseded by hydrants or other form of faucet placed at the edge of the sidewalks; they are generally a little column, with a faucet at the top. The fire hydrants are to be found at the corners of the streets. In the public gardens and parks the use of water is also gratuitous and sufficient for irrigation and the service of ornamental fountains. The arts make considerable use of the waters, there

* A localism explained in the context.—*Trans.*

being many establishments for the manufacture of articles of food and domestic use, while there are many manufacturers of cloths, hats and other subjects of the national industry using the water for their steam motors.

Conclusion.—This résumé has been brief, purposing only to give an idea of the service and distribution of water in the city of Mexico. The growth and importance of the city increases daily, and the quantity of water required for its use therefore increases. Thus it is that the Municipal Council, impressed with the necessity for fulfilling its obligations to the public health in the abundant supply of pure water, neglects no sacrifice, expense nor enterprise that has for its object the improvement of this important branch of the public service.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

650.

(Vol. XXX.—November, 1893.)

PURIFICATION OF SEWAGE AND OF WATER BY FILTRATION.

By HIRAM F. MILLS, A. M., C. E.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

Six years ago the State Board of Health of Massachusetts began experiments to supply the deficiency that was found to exist in the knowledge of the engineering profession upon the subject of filtration of water and of sewage. The ends in view were the prevention of pollution of streams, the purification of domestic water supplies and the general preservation and improvement of the public health.

The objects were not to make a better-looking water or one that would be better adapted for use in steam-boilers or for certain manufacturing purposes, but to learn the best methods of removing those impurities in drinking water that may be injurious to health, and, if

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

possible, to purify sewage so that the effluent may with safety be turned into a drinking-water stream.

It appeared probable, from the experiments which had been made by Frankland and Warington, in England, and by Schlœsing, in France, and at the Clichy Laboratory, near Paris, that these objects could be accomplished by intermittent filtration. Their experiments, few in number, but giving astonishing results, were made in 2-in. glass tubes, in glass cylinders 10½ ins. in diameter and about 5 ft. long, and in a glass tank 6½ ft. high and 8 ins. square. Diligent search at that time failed to find other experiments in which the chemical character of the effluent could be definitely compared with that of the applied sewage.

Excellent effluents had been obtained in the experiments on irrigation by the British Association for the Advancement of Science, and by Marié-Davy at Paris, and on irrigation fields in Germany, which might now be attributed to filtration ; but their purity was then regarded as depending to a great degree upon the action of growing crops. In these results Marié-Davy had shown a remarkable purification from bacteria.

There were, in 1887, various filter beds in use in England and on the Continent, mostly in connection with irrigation fields, or used, themselves, for growing crops. They were receiving from 36 000 to 90 000 galls. of sewage per acre per day, and were giving results satisfactory to the operators ; but the few published results of analyses of the effluents were less satisfactory than those obtained in the laboratories.

There was not sufficient knowledge upon the subject in the profession to enable a well-educated engineer to decide whether any selected area would purify sewage applied to it, and, if it would purify, the amount to be applied and the proper intervals for application, and whether and under what conditions disease-producing germs could be removed were unknown.

To supply knowledge upon these points and others pertaining to the purity of water supplies, the Experiment Station of the State Board of Health of Massachusetts was established at Lawrence in 1887. Some of the more important general results of the six years of careful investigation will now be presented.

A general description of the Station and the methods of investiga-

tion carried on there, together with the earlier results obtained, may be found in the published report on water supply and sewerage, entitled, "Experimental Investigations by the State Board of Health of Massachusetts upon the Purification of Sewage by Filtration and by Chemical Precipitation, and upon the Intermittent Filtration of Water. Made at Lawrence, Mass., 1888-90." Additional results may be found in the more recent "Annual Reports of the State Board of Health of Massachusetts."

A model of the Experiment Station, and samples of some of the sands used there, together with diagrams of some of the earlier results and samples of the sewage and of the effluents, may be seen in the south-eastern portion of the floor of the Anthropological Building, which is at the south-eastern extremity of the Fair Grounds.

Some idea of the extent of the laboratory work in the Department of Filtration may be conveyed by the following statistics.

There have been analyzed chemically :

1 700 samples of sewage applied to filters,
6 400 samples of effluent from filtered sewage,
400 samples of water applied to filters, and
970 samples of filtered water ; also,
611 samples of substances applied to filters and flowing from them, for the purpose of determining the effect of filtration upon various substances contained in sewage, making a total of 10 081 chemical analyses of samples connected with filtration.

Bacterial samples of sewage and effluent corresponding with 6 770 of the above chemical samples were examined, and about 4 000 such samples which were not examined chemically. Nine thousand other bacterial samples were examined in connection with filtration.

Samples of sand used in filters examined for bacteria numbered 880, making a total of 20 650 bacterial examinations connected with filtration. There have been made 3 700 comparative species determinations of the bacteria in water before and after filtration.

When this work was commenced it was definitely known that no nitrification, or complete oxidation of nitrogenous organic matter, would take place without a current of electricity or the presence and co-operation of organic life. Robert Warington had found by numer-

ous ingenious experiments that nitrifying organisms were to be found abundantly in soil, and it had been concluded that soil was essential to a filter-bed; but the experiments of the State Board of Health show that nitrifying organisms are contained in the several kinds of drinking-water supplies of all parts of the State and probably in all sewage, and that into filter-beds of sand or gravel containing no soil they are after a short time introduced in sufficient numbers by the sewage; hence, that soil is unnecessary for this purpose.

Organic life alone cannot produce nitrification; there must be present free oxygen. Unlike most drinking waters, sewage contains no free oxygen, if any air enters it, the oxygen is absorbed by the carbon of the organic matter; and in order that nitrification may take place there must be provided an excess of oxygen beyond that which will be readily taken up by the carbon.

The circumstances that bring most clearly to mind the essential conditions of intermittent filtration are these: a bed of gravel stones, as large as robins' eggs, having an abundance of nitrifying organisms attached to each stone, has sewage poured over it for a short time; after the sewage has settled away, every stone it has reached is covered with a thin film of liquid in contact with air and enveloping the nitrifying organisms. Then nitrification takes place rapidly and with great completeness. A single hour will make a marked change in the character of the adhering liquid.

After a few hours another charge of sewage distributed over the filter will mingle somewhat with the former adhering liquid and cause it to move downward. If this charge is too abundant, a part of it may flow too rapidly through the large interstices between the stones and reach the outlet drains without being completely nitrified. This condition limits the amount that can be purified by coarse materials. The air spaces are larger than necessary, and the area of surface is small. By decreasing the diameter of the stones to one-tenth of their present diameter, we may still have the same amount of air and water space and ten times the surface to which the water may adhere. In this case the air spaces between the films, though very much smaller than before, are sufficient to supply the oxygen necessary for nitrification if we allow sufficient time between applications for the sand to become drained. It is, however, evident that if we continue diminishing the diameter of the grains of material, we shall reach a condition in which

the films of water adhering to the grains will occupy the whole space; there is then no room for air, and nitrification cannot take place. Sand in a filter-bed that remains saturated after draining 24 hours is of little value and may be very objectionable.

Up to the present time the experiments of the State Board of Health indicate that the sand of even grain that presents all of the conditions most favorable for the very complete purification of sewage has a diameter of grain of about two-tenths of a millimeter, or about eight-thousandths of an inch.

The more even in size the grains are the better, but as we rarely find a sand in which some of the grains are not many times the size of others, and as the conditions for nitrification depend more essentially upon the finer grains of the mass, we have found it more intelligible to classify sands by the finer 10% of their grains, and in thus classifying sands we should say, that sand presents the conditions most favorable for very complete purification of sewage, the finer 10% of whose grains has diameters equal to and less, but not much less, than 0.2 mm.

In soils, the organic matter, which is much finer than this, makes up a considerable part of the finer 10% of the particles, and this, sticking to the other particles, decreases the air and water spaces to a greater extent than would appear from the mechanical analysis; hence, while a filter-bed made of soil, whose finer 10% has diameters equal to and less than $\frac{2}{100}$ mm., will be entirely ineffectual in purifying the smallest amount of sewage, one made of clean sand, whose finer 10% has diameters equal to and less than $\frac{3}{100}$ mm., may satisfactorily purify for years 30 000 galls. of sewage per acre daily.

With the kind of sand indicated above as being most favorable for very completely purifying the largest amount of sewage, the quantity that may be permanently applied to filters having a depth of 5 ft. of such sand is found to be about 100 000 galls. per acre daily.

The very complete purification here referred to produces an effluent chemically as good as many drinking waters, having in 100 000 parts about 0.0020 part of free ammonia, and about 0.0130 part of albuminoid ammonia, the former being but $\frac{1}{10}$ of 1%, and the latter but 2%, of the amount in the sewage. Nearly all of the organic matter of the sewage is burned up, forming in the effluent soluble nitrates to the extent of about 1.8 parts in 100 000. Bacteriologically, the results are

even more remarkable, there remaining in the effluent less than 1 in 10 000 of the number applied in the sewage. The appearance of the effluent is that of a bright, clear spring water.

With finer sands similar results may be obtained with decreasing quantities, one-half of the quantity being filtered by sands of whose finer 10% the maximum grains have diameters of $\frac{5}{100}$ mm., and excellent purification may be obtained with sands so fine as to allow the application of only 25 000 galls. per acre per day. In fact, with the finer sands, having a proper daily charge, the bacteria of the sewage may be more completely removed than with the sand allowing the greatest amount to be purified chemically.

With sands coarser than the most favorable size, similar results chemically have been obtained with diminished quantities, one-half the maximum quantity being filtered by sands of whose finer 10% the maximum grains have diameters of $\frac{5}{10}$ mm. Bacteriologically, these coarser sands do not give so good results, but they remove more than 99% of the applied bacteria. Sands coarser than that given as most favorable may be used for less perfect results for the full quantity of 100 000 galls. per acre per day up to the larger size just mentioned.

With still coarser sands and open gravel the quantity of sewage that may be purified for a time may be greater than the maximum amount given above, if the periods of application be made sufficiently short and sufficiently frequent; but after a time such treatment causes a thick coating of slime to form upon the surface of the grains and stones at a considerable depth, where the air is not changed rapidly enough to favor complete nitrification; hence, it is necessary to treat such coarse materials with less than the maximum amount, or else resort to drawing or forcing air through the filter, which expedient we have found capable of increasing the nitrification and allowing a much larger amount of sewage to be nitrified in coarse sand filters.

With the material best adapted to the purification of large quantities of sewage, there is a deposit of nitrogenous matter in the upper layers amounting to about 15% of all of this kind of matter applied in the sewage. The other 85% is oxidized and passes off in solution as nitrates in the effluent. The effect of this deposit on and near the surface is to decrease the air space between the grains of sand and increase the amount of water continually held by these layers; thus, in time preventing sufficient change of air in the layers beneath. Continued

efficient action may be had by frequently raking over the surface to the depth of 1 in.; but there comes a time when this upper inch should be removed and replaced by clean sand. With the full complement of 100 000 galls. per acre daily of such sewage as has been filtered at Lawrence, the upper layer 1 in. in depth should be replaced by clean sand once in about eight months. With other rates of application of sewage, about 1 000 galls. of sand should be replaced for each 1 000 000 galls. of sewage applied.

The sewage that has been used upon the filters of the Experiment Station at Lawrence is pumped from one of the main sewers above where manufacturing wastes enter, and where the sewage, coming from stores and from the homes of about 10 000 people, may be regarded as ordinary city sewage, including storm water. The sewage is drawn through a cylindrical strainer lying horizontally and diagonally across the sewer a few inches above the bottom. This strainer has many holes $\frac{3}{4}$ in. in diameter. Lying as it does between the bottom and the top of the stream of sewage, it is probable that the sewage drawn through the pipe does not contain as much of the fatty substances floating on the surface of such a stream, nor as much of the mineral matter moving over the bottom as if the sewage flowed directly from the mouth of the sewer upon the filter, but quite as much of these substances as sewage separate from storm water passed through grease traps and the settling basins usually used with sewage to be filtered.

An average analysis of the sewage put upon the filters in parts per 100 000 would be about as follows: Loss on ignition, 20 parts; fixed residue, 30 parts; free ammonia, 2 parts; albuminoid ammonia, 0.7 part, and chlorine, 6 parts. One-third of the organic matter shown by loss on ignition is in suspension. One-fifth of the mineral matter and one-half of the nitrogenous matter as shown by the albuminoid ammonia is in suspension. The average number of bacteria was 800 000 per cubic centimeter.

With sewage containing more nitrogenous matter in suspension than here given, the upper inch in depth of sand would require to be replaced more frequently than above stated, and with correspondingly less frequency if the nitrogenous matter in suspension were less.

With such sewage as above described an excellent but attainable result during warm weather is an effluent whose analysis has been

already given and is now repeated for immediate comparison. Free ammonia, 0.0020 part; albuminoid ammonia, 0.0130 part, and nitrogen as nitrates, 1.8 parts, and less than $\frac{1}{100}$ of 1% of the number of bacteria applied.

To obtain as good a result in a cold New England winter it is necessary to protect the surface of the filter-bed so that it will not freeze. If the sand freezes at the surface the sewage thaws places here and there and passes too rapidly through the sand in sections having less than the whole area of the bed ; air is partially excluded and nitrification is checked. A closed shed which will keep snow and wind from the bed will be sufficient covering where the sewage used has the ordinary temperature of sewage in sewers of 45° Fahr. or more.

Unprotected beds treated with such sewage may be expected to have nitrification very much checked and an increase in organic matter in the effluent shown by the albuminoid ammonia, amounting to three times the quantity found during the warm months, with a larger multiple of the bacteria. Four such beds through the coldest months removed an average of 91% of the albuminoid ammonia of the sewage and 97% of the bacteria ; but there results from the decrease in nitrification a storing of organic matter in the filter which is only partially removed in the warm months, and the permanence of the filter is somewhat impaired. If filters are to be run nearly to their full capacity, they should be protected from freezing.

Fine sand filters may have their capacity increased by excavating trenches about 5 ft. apart, 1 ft. wide and 2 ft. deep, and filling them to within 3 ins. of the top with coarser sand and applying all of the sewage to the surface of the coarse sand. In winter, boards may be put on across the trenches under which the sewage will flow and, being protected from snow, it will not freeze. Sand so fine that 15 000 galls. per acre per day exceeds its capacity may thus be enabled to filter satisfactorily 30 000 galls.

The amount of sewage that may be applied at one time depends upon the size of the grains of sand and the resulting amount of air and of water held after draining. With gravel stones which retain very little water the amount applied at once should not be as great as the amount retained, otherwise some of it will flow through immediately without sufficient time for the chemical changes to take place. On the other hand, fine sands retaining a large amount of water hold but little

air; and as sewage needs as much as its own volume of air for its oxidation, we have here the quantity that may be applied at one time limited by the quantity of air held by the filter when the sewage is applied.

The quantities that have been indicated as being very completely purified daily by the different sands, express nearly the maximum quantity that may advantageously be applied at once. Smaller quantities, with intervals of a fraction of a day, may be advantageously applied to coarse materials.

As illustrated by the filter of gravel stones, the purification of sewage by intermittent filtration is in no essential degree a straining process; it is essentially exposing the liquid for a sufficient time in thin films upon the surfaces of solid particles having nitrifying bacteria adherent, with air in contact with the films. These are the conditions most favorable for the chemical change, which consists of burning up the organic matter and producing the harmless mineral matter which we have called the nitrates. In continuous filtration these conditions do not exist, and no such purification is possible. It is equally true that the removal of bacteria from the liquid depends only incidentally upon straining; for example, upon straining sewage through filter paper, which is supposed to remove all suspended matter, the number of bacteria remaining in the effluent was 20% of the number applied, while the number leaving a filter of gravel stones 5 ft. deep was less than 1% of the number applied when filtering intermittently 80 000 galls. of sewage per acre daily.

Bacteria are killed by subjecting them to conditions on the passage through the filter unfavorable to their life. One very important condition is depriving them, for a sufficient time, of suitable food material. This is most effectually accomplished by burning up the organic matter in the process of complete nitrification; so that, in general, the conditions which produce the most complete chemical purification are those most favorable for the removal of bacteria. It also results that the effluent in which nitrification has been complete is unfitted to support the life of bacteria. On the other hand, water that has been boiled supplies a good food for bacteria, and if such water be treated with a small percentage of unboiled water, the bacteria thus introduced may increase in a few days to a thousand times the number then to be found in the unboiled water. If, when very completely nitrifying the

sewage applied to it, a filter be flooded with water from a boiler, the following effluents from sewage will contain a much larger number of bacteria than usual.

It was found that heating the sand of a filter to 300° Fahr., or more, and pouring boiling water through it caused 100 times as many bacteria to pass through it with sewage for three months as passed through a similar filter whose sand and first water had not been heated. This appears to be due to preparing the organic matter in the sand and in the water by heat, so that it is a better food for the bacteria.

When working along with steady application of ordinary sewage, with surface kept sufficiently free to allow air to enter and make nitrification satisfactory, the number of bacteria found in the effluents of 10 filters averaged $\frac{5}{1000}$ of 1% of the number applied in the sewage. With filters having sands coarser than the sand which we have specified as best adapted to purify the largest amount of sewage, the number remaining in the effluents was about $\frac{2}{10}$ of 1%; while from the filters with sands finer than the standard, the number remaining in the effluents was but $\frac{1}{1000}$ of 1% of the number applied in the sewage. In fact, we have the strongest reason to conclude that with these sands no bacteria ordinarily lived through the passage, but that under peculiar circumstances a few survived, and these lived in the under-drains, and occasionally from these a few units came in the effluent to represent the hundreds of thousands applied. Sometimes the few found in the effluent were a pure culture of a single species of bacteria.

With the facts that with the coarser sands the number of bacteria that live to get through the filter, when nitrification is nearly complete, is an extremely small percentage of the number applied in the sewage, and that the effluents from highly nitrifying filters are not adapted to the support of bacteria, we have still to ask whether any disease germs will pass such filters, and if any pass, will they increase in the effluent and in a stream to which this may flow?

It has been found that most of the bacteria that get through the filters are of kinds that live and are propagated in wet sands and underdrains. If typhoid fever germs were applied with the sewage, their lateness of development upon gelatine plates allows other bacteria to grow in such numbers or to liquify the plates to such an extent that the typhoid fever germs, if present, can hardly be distinguished; hence, it becomes necessary to select a foreign species, not

found in our sewage, having habits of life similar to typhoid fever germs, that can be readily distinguished from other bacteria; such is *Bacillus prodigiosus* which has been cultivated in large numbers and applied by the millions to the several filters.

With sands whose finer 10% had grains as large as $\frac{5}{10}$ mm. we have succeeded in passing through a few of the *Bacillus prodigiosus*, and conclude that typhoid fever germs would pass through such sands in very much reduced numbers.

With sands of $\frac{3}{10}$ mm. and less we have not succeeded in finding in the effluents any of the large numbers of *Bacillus prodigiosus* applied with the sewage. These results, together with the fact that typhoid fever germs continually decrease in numbers, not only in the effluents from these filters, but even, although less rapidly, in sewage-polluted waters which have not been filtered, dying out in the latter in from one to three weeks, give us confidence in concluding that filters made with these finer sands, so long as they are kept in a condition to very completely nitrify the sewage applied, will give an effluent that may with safety be turned into a drinking-water stream.

Study of the working of intermittent filters shows them to be not mechanical strainers, but delicate organisms gradually growing fitted to perform the work required of them, needing time to adapt themselves to any marked change in the amount or quality of the organic matter applied to them, but regularly, and for many years, performing an astonishing amount of chemical and biological purification, if the essential conditions for their action are maintained.

As previously stated, the sewage pumped from the Lawrence sewer does not to any large extent contain wastes from manufactories and dye works. Sewage highly colored at the Station with the ordinary coloring matters used in dye works was applied intermittently for weeks to several filters with the resulting effluent entirely colorless.

Acid sewage, applied occasionally, does not interrupt nitrification, but if long continued, with sand containing no alkali, it causes nitrification to cease. This may be prevented and nitrification continued indefinitely if in constructing the filter thin layers of granulated limestone, having grains about the size of the sand grains, are built into the upper part of the filter.

Soap applied in large quantities may clog the upper layers of a filter and prevent nitrification, in which case the upper layers should be replaced by clean sand.

Filtration of Water.—The principles of the purification of sewage by intermittent filtration are applicable to the purification of polluted water supplies, with this important difference: water supplies have not sufficient organic matter to combine readily with all of the oxygen the water absorbs from the air, consequently they contain more or less free oxygen; hence, they do not require so frequent renewal of air in the sand, and much larger quantities can be passed through the filter and find there a sufficient additional amount of air to combine with what nitrogenous matter they contain, to nitrify it through longer periods of continuous use than the still more polluted water which we call sewage.

We may in a very general way distinguish between the two by saying sewage contains 50 times as much nitrogenous organic matter as a polluted water supply. The question may then arise, can we satisfactorily purify by filtration 50 times as much water as sewage upon the same area? It may prove to be practicable in the future, but at present we will give attention to results that have been obtained when filtering about half of this amount.

One of the first steps in considering what sand to use in a filter is to determine how much water will flow through it with the available head. Mr. Allen Hazen, the chemist in charge of the Experiment Station, found the quantity that would flow through the sands ordinarily used for filtration* to be, at a constant temperature, directly proportional to the acting head, and inversely proportional to the

* Mr. Hazen also made a few experiments to aid in determining the amount of coarse material necessary for underdraining the sand, which may be summarized as follows:

QUANTITY OF WATER FLOWING THROUGH GRAVEL STONES AT DIFFERENT HEADS.

Loss of head in terms of distance through gravel.	Size of gravel stones in millimeters and inches. The finer 10% finer than—							
Millimeters	3	5	8	10	20	30	40	50
Inches.....	0.12	0.20	0.32	0.39	0.79	1.2	1.6	2.0
	Quantity flowing in 1 000 000 gallons per acre daily.							
0.0005	.....	4	12	22	35	90	170	280	410
0.001	.....	7	23	43	65	160	300	480	690
0.002	.....	15	43	84	120	300	490	720	1 040
0.004	.....	29	83	160	230	520	790	1 070	1 360
0.01	.....	72	185	320	420	890	1 280	1 670	2 150
0.02	.....	136	320	500	630	1 280	1 800	2 340	3 030
0.05	.....	300	600	950	1 140	2 030	2 862	3 734	4 800
0.10	.....	530	1 000	1 400	1 650	2 850	4 044	5 280	6 800

depth or distance through the sand. He also found it directly proportional to the square of the maximum diameter of the finer 10% of the sand grains. Calling the maximum diameter in millimeters d, the head in feet h, the distance flowing through the sand l, and the quantity in 1 000 000 galls. per acre per day Q, he found the following formula to satisfactorily express the results of his experiments with water at about 50° Fahr.:

$$Q = \frac{800\ h\ d^2}{l}.$$

The sand which we have found capable of very completely purifying the largest quantity of sewage having d equal to $\frac{2}{10}$ mm. would, when 1 ft. deep and with 1 ft. acting head, allow 32 000 000 galls. per acre to flow through it in 24 hours; and the same quantity would flow through any other depth if the acting head were equal to the depth. There are practical considerations that make it desirable to depend upon a less head than the distance the water passes through the sand. If there be 1 ft. acting head for a passage of 5 ft. through the sand, the maximum quantity that would flow through this sand in 24 hours would be 6 400 000 galls. per acre; but the whole 24 hours would not be available if the water be applied intermittently. With the Merrimac River water used in the experiments it was found that filling the sand with air once a day was sufficient; in which case we can count upon a steady flow through the sand for two-thirds of the 24 hours, thus passing 4 300 000 galls. per acre per day when the sand is clean; but this quantity would be still further reduced, for the water has some organic matter and some mineral matter in suspension at all times, and during the spring freshets the amount of mineral matter is very large. All of this suspended matter tends to clog the interstices of the upper layer of the filter and to allow less water to pass; hence, the sand of the filter must be coarse enough to allow the desired quantity to flow through when the upper layer is obstructed, and not make the intervals between scrapings of the surface to remove this deposit too inconveniently short.

With the water used, the sand we are considering allowed 60 000 000 galls. of water to pass through 1 acre between the times when it was necessary to scrape the surface and remove about $\frac{1}{8}$ in. of sand and sediment and replace with clean sand.

When filtering 2 000 000 galls. per acre daily, the deposit had to be

removed once a month in the six months from May to October, and more frequently during the spring and fall freshets. With coarser and finer sand the amount passed between scrapings was nearly in proportion to the size of the sand grains.

Sands, the larger grains of the finer 10% of which are from $\frac{1}{10}$ to $\frac{3}{10}$ mm. in diameter, and whose capacity to pass water through a depth equal to the acting head ranges from 8 000 000 to 72 000 000 galls. per acre in 24 hours, have given excellent results when filtering 2 000 000 galls. per acre per day, and the mean of these sizes has also satisfactorily purified 3 000 000 galls. per acre per day.

With still coarser sands, up to $\frac{5}{10}$ mm. in diameter, good results have been obtained when filtering 1 000 000 galls. a day; but the results were variable, and the largest size mentioned has not given assurance that sewage polluted water can be constantly purified from its disease germs.

With filters having 5 ft. in depth of sand, whose finer 10% had maximum grains from $\frac{1}{10}$ to $\frac{2}{10}$ mm. in diameter, the chemical purification, when filtering at the rate of 1 250 000 to 2 000 000 galls. in 24 hours, was very satisfactory, both with continuous and with intermittent filtration. The following are average results in parts per 100 000 :

	Free ammonia.	Albuminoid ammonia.	Nitrogen as nitrates.
Unfiltered water from the river	0.0070	0.0187	0.0119
River water filtered continuously	0.0008	0.0078	0.0251
River water filtered intermittently	0.0007	0.0075	0.0468

As before indicated, there is in drinking water nearly enough absorbed oxygen to combine with the nitrogenous organic matter to burn it up or to nitrify it, when it is brought in contact with nitrifying bacteria under conditions favorable for their action. Filling the sand with air once a day furnishes the additional quantity of oxygen necessary for very complete nitrification.

In the results just given we find in continuous filtration the free ammonia is reduced from 0.0070 to 0.0008 part, or to 11% of the original; the albuminoid ammonia is reduced from 0.0187 to 0.0078 part, or to 42%, while the nitrates are more than doubled. In intermittent filtration the free and albuminoid ammonias are reduced to 10% and 40%, respectively, and the nitrates are quadrupled. The large additional amount of the nitrogen which has come through the intermittent filter as soluble nitrates has remained in the continuous

filter and often gives rise there to growths of organic matter which are objectionable.

Similar chemical results may be obtained with filters having sand 4ft. deep. With less depths the purification is less complete, but even with a depth of only 1 ft. of sand the free and albuminoid ammonia were reduced to 21% and 52%, respectively, of the unfiltered water, and the nitrates were nearly doubled; still, these shallow depths cannot be depended upon to give constantly reliable results.

While the chemical change that takes place in the water when passing through the filter is important, it is much less important than the removal of bacteria. One very important effect of burning up the organic matter and forming nitrates, is removing the food supply that would support the life of injurious bacteria. It is probably fair to state that the invisible vegetable kingdom of bacteria is as a whole no more injurious to man than the whole of the visible vegetable kingdom. As in the latter, there are certain poisonous plants, so in the former there are a few species that cause disease in man. Of these we know but little, but as far as studied we do not find that they increase in sewage polluted water. The temperature and food supply they find in the human body are better adapted to their propagation, and while they continue for a time to exist when turned with sewage into a stream or pond, it is probable that they continually decrease in numbers.

The longest time we have been able to keep the germs of typhoid fever alive in Merrimac River water is about three weeks ; more commonly they disappear in one week. This short period of existence of typhoid fever germs in such waters presents the probable reason why typhoid fever may be readily carried down a river from city to city using the water for drinking, while a polluted stream may enter one end of a large pond whose waters are changed only after months, and a water supply drawn from the opposite end may be continually free from the disease pollution. It is also a reason why we may be satisfied in removing nearly all, if not quite all, of the disease germs from a polluted water by filtration.

As there are found to be certain species of water bacteria that grow and increase in numbers in wet sand and in the under drains of filters, the efficiency of filters in removing disease germs must be determined by applying to them water inoculated with a known number of these

germs or with others having similar conditions of life, and learning how many, if any such, remain in the effluent.

Most of the tests which the State Board of Health relies upon as giving the efficiency of different filters in removing disease germs, have been made by applying to the filters water containing large numbers of the typhoid bacillus or of the more easily recognized *Bacillus prodigiosus*.

These tests show that with sands whose finer 10% have grains equal to or less than from $\frac{1}{10}$ to $\frac{2}{10}$ mm. from 1 500 000 to 3 000 000 galls. of water containing large numbers of these germs may be filtered daily upon an acre with the constant removal of more than 99½% of the germs, and it is probable that a similar result would attend the use of sand up to a diameter of $\frac{3}{10}$ mm.

It is found that after scraping the surface of the filter, a few more germs are found in the effluent than before, but the number removed at this time is still more than 99½% of those applied.

With the intelligent care and favorable conditions at the Experiment Station less than ½ of 1% of the applied germs live to get through the water filters, with sands varying in depth from 1 ft. to 5 ft.; and, within this limit, the deeper filters and the finer sands allow smaller numbers to pass.

The difference in the efficiency of removing these applied germs with sand 1 ft. deep, or with sand 5 ft. deep, is so small that it is evident that nearly all of the removal is effected by the upper foot of material. In fact, in a filter having 1 ft. in depth of sand which had been filtering continuously for six months the surfaced was scraped, and in the next ten days more than 2 000 million of *Bacillus prodigiosus* were applied from day to day. The sand was then carefully examined and found to contain 13% of the number applied, 93% of which were found in the upper ¼ in. None were found alive below the upper inch. This was not the case with the ordinary river bacteria; they were distributed throughout the sand, but with greatly decreased numbers below the upper inch.

With intermittent filtration of water, the number of germs remaining alive in the sand is very much less than with continuous filtration. Some examinations in the summer have shown the number in the upper ¼ in. to be but 1% of the number that had been applied. The ordinary river bacteria do not survive within the sand in intermittent filtration to nearly the same extent as in continuous filtration.

Long series of experiments with the filters which we regard as adapted to the filtration of water supplies have shown, immediately after, as well as before, scraping the surface, a removal of more than $99\frac{5}{10}\%$ of the applied germs, and if all of the disease germs like those of typhoid fever do not increase, but rapidly decrease, in filtered water, we may conclude that the $\frac{5}{10}$ of 1% of the number applied will do little or no harm.

Such results can only be obtained in actual practice by having the filters constructed with the most intelligent care, having in view particularly that the sand shall be so placed that it forms a homogeneous mass without stratification or coarse-grained channels down through which an undue quantity of water may flow ; and when in use the surface should be as intelligently kept in order so that every square foot shall continually do its allotted part of the work.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

651.

(Vol. XXX.—November, 1893.)

THE SEWERAGE SYSTEM OF MILWAUKEE AND THE MILWAUKEE RIVER FLUSHING WORKS.

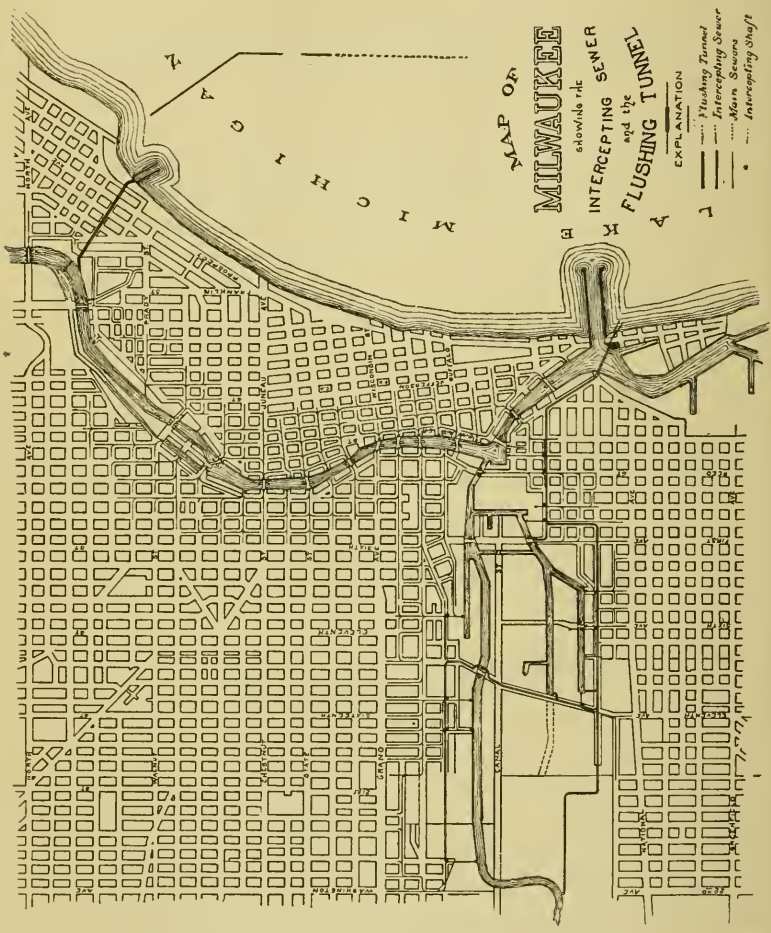
By G. H. BENZENBERG, M. Am. Soc. C. E.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

The city of Milwaukee, with a population of about 240 000, is situated on the west shore of Lake Michigan about 100 miles below the head of the lake and 86 miles north of the city of Chicago. Its area is 21 sq. miles, having a length of 6½ miles north and south along the lake front, with a maximum width of 4½ miles from the lake shore. It is divided into four natural divisions by three rivers converging into one common outlet about midway between the north and south limits. The natural outlet of these rivers was originally ½ mile further south, but, in order to improve the harbor, the city in 1854 constructed the present outlet, which since that time has been ex-

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

tended and maintained by the Government as the harbor entrance, the old outlet meanwhile having been closed by the sand accretions and the action of the waves.



By enactment the city is divided into three divisions. The east division, having an area of $3\frac{1}{2}$ sq. miles, is situated between the lake and the Milwaukee River, which flows from the north, nearly parallel to the lake shore and from $\frac{1}{2}$ to 1 mile distant therefrom; the west division, with an area of $9\frac{1}{2}$ sq. miles, is situated west of the Milwaukee and north of the Menomonee River, which flows from the west and

joins the Milwaukee River about ½ mile west from its outlet; the south division, with an area of 7¾ sq. miles, is situated south of the harbor and the Menomonee River, and lies on both sides of the Kinnickinnic River, which is the smaller of these streams and flows from the southwest emptying into the Milwaukee River at its present outlet. The lower east and west divisions became first settled, and form to-day, for a distance of ½ mile north of their southern boundary, the center of the business section as well as of the population of the city. Near the mouth of the rivers, which are each navigable and docked for a distance of about 2½ miles, the territory was originally covered by a large marsh which now forms the low level district, and from which point the ground gradually ascends, the river banks becoming steeper in proportion to their distance from their outlets until the ground finally reaches an elevation of from 100 to 140 ft. above the lake level. The various earlier manufactories were originally located along the banks of the Milwaukee River, but have since, with the exception of the tanneries, been removed to the Menomonee and Kinnickinnic valleys, which, on account of their greater width, have become the great railroad yards and their outlets. The natural flow in these rivers is, excepting in the spring or at the time of a long, severe rain storm, very small, at times in the summer ceasing altogether. The flow over the dam across the Milwaukee River, originally constructed 3½ miles above the harbor entrance for canal and power purposes, averages during the summer months not over 15 000 000 galls. per day, notwithstanding the watershed of this river covers an area of 675 sq. miles. Hence, the current in these streams is naturally very sluggish, and, on account of the winds on the lake, nearly as often up stream as down stream.

In 1869, when the population had reached 68 000, a general sewerage system, designed by the late Mr. E. L. Chesbrough, M. Am. Soc. C. E., was adopted, and the construction of sewers according to such system commenced on a large scale. This system, which covered a territory of 4¾ sq. miles in the heart of the city, contemplated the removal of all storm water as well as sewage, and provided for the discharging of the same into the streams at the banks of the rivers. The plan of this system has since been extended and added to until now it covers the entire present area of the city. There have been constructed 226.41 miles of sewers, at a total aggregate cost of $2 888 977.71.

Forty-seven or 63% of these sewer outlets, varying from 18 to 96

ins. in diameter, are located along the Milwaukee River, and drain 40% of the above area into it. Forty-two per cent. of the area is drained through 17 sewers into the Menomonee valley, in addition to which the stream and canals in said valley also receive the refuse of all the large packing-houses, distilleries and other like establishments located along their banks.

In 1872 the construction of the water-works of the city, designed by the late Moses Lane, M. Am. Soc. C. E., was commenced. These have been extended, until now the total cost of the works has reached the sum of $4 013 994.46, and the distribution system covers a total length of 229.14 miles of water mains. The introduction of the water-works naturally hastened the extension of the sewerage system. The result of the discharge of the sewage and factory refuse into the sluggish streams soon became noticeable, so much so, that within 10 years after the adoption of the system, the offensiveness of the rivers and canals in the Milwaukee and Menomonee valleys became a matter of public agitation and earnest complaint. Intercepting sewers along the banks of all the rivers were generally suggested as the proper remedy. To determine the best method of abating the ever-increasing nuisance, and to insure the purity of the water supply, the solution thereof was entrusted to an engineering commission, consisting of Messrs. E. S. Chesbrough, Moses Lane and George E. Waring, Jr., who in 1880 recommended the purchase of 500 acres of land whereupon in the future to dispose the sewage by irrigation. They also recommended the construction of a series of low-level intercepting sewers, the contents of which were to be pumped out into the lake 1 mile south of the harbor entrance, as well as the construction of self-acting flush tanks at the heads of the sewers.

As a result of these recommendations, a plan providing for the construction of an intercepting sewer from the lake shore immediately south of the harbor, westward and to the south of the Milwaukee and Menomonee Rivers to the center of the large packing-house district, was adopted; and in 1881 the east 4 300 ft. of this sewer, being from the west bank of the Milwaukee River westward, was put under construction. Succeeding to his present office in 1882, the writer believed that to even partially restore the waters of the Menomonee valley to their former condition would require, not only the collection of the refuse of the factories, but the intercepting of all the sewers of the

system emptying thereinto, and principally the creation of a flow in the terminal canals and in the sluggish stream itself by artificial means, by the constant introduction of a body of purer water to assist in the purification by dilution. Hence, the size and grade of the unfinished part of this intercepting sewer, as well as its location, was changed, so as to keep it entirely below the surface of the canals, by extending it so as to reach every sewer outlet on each side of said valley. The amended plan provided for submerged connections with the upper end of each terminal canal, and also at intermediate points, so that by using the excess capacity of the intercepting sewers to remove the stagnation of the canals, purer water would follow, which again would aid in diluting the crude sewage in the sewer on its way to the lake, thus constantly increasing the dilution and assisting Nature in the performance of its restorative functions. This natural, healthy and universal process of purification of all decaying matter is continuously and uninterruptedly going on everywhere in life, either in the air, water or in the earth, so long as stagnation is not permitted to clog the process, and the healthy life germs, whatever form they may take, predominate in sufficient proportions. Herein undoubtedly lies the solution for the abatement of all nuisances from whatever source they may originate, or for the destruction of all disease germs. Nature has amply provided for the performance of this function everywhere, excepting where man has interfered therewith by accumulation or by a neglect of natural laws. To avoid and escape the results which quickly follow and augment upon such interference, man must assist in restoring such natural process, and, if caused by accumulation, must, by a proper redistribution or dilution, sufficiently re-establish the conditions necessary to the resumption of the process of purification; generally this may be successfully accomplished by artificial means, providing the proper conditions are maintained. Wherever the artificial process of purification has failed, it has generally been due to the fact of not understanding or of not maintaining the proper distribution of decaying matter, either in irrigation, filtration or dilution, or in not following the laws of Nature.

With this apparent fact in view, the uncompleted part of the intercepting sewer for the Menomonee valley was enlarged generally to the dimensions of the completed part, which was of 5 ft. 4 ins. internal diameter, although 7 ft. diameter would have been a more suitable size, and

in accordance with this amended plan the work has been completed. The first section of this sewer was constructed almost entirely of 8-in. timber beveled, to assume the cylindrical form; all subsequent work, however, was constructed of brick. There are two river crossings consisting of wrought-iron inverted siphons. The first, near the outlet, carrying the sewage across the Milwaukee River to the pump well, was laid in 1884, is 50 ins. in diameter and 520 ft. in length, and laid 26½ ft. below the river surface, connecting steel and brick-lined shafts on each side of the river. The second was laid in 1888, and crosses the North Menomonee Channel at a depth of 24 ft. below the water surface; is 40 ins. in diameter and 200 ft. in length. This siphon has unequal ascending arms, forms several slight angles in its crossing so as to avoid a bridge pier, and at its upper end has a submerged connection with the channel, for the purpose of flushing out the siphon and overcoming the stagnation of said channel.

Ninety miles of sewers now discharge their sewage into this interceptor through 12 connections. These connections are so constructed as to permit the dry weather flow, or sewage proper, to flow through an opening into the intercepting sewer below, over which the storm water is carried towards the outlet of the sewer into the streams by virtue of the increase of velocity due to such increased volume. This style of connection, which needs no attention or mechanism to regulate the flow of the sewage, can be easily constructed to perform its duty accurately. Plate I shows a section of the first of these connections, constructed in 1887, which has required no attention since, though it has been frequently examined as to its operation.

The submerged connections with the canals are each controlled by a valve, so as to regulate the flow from these canals, both as to volume and location, always utilizing the intercepting sewer to its greatest capacity by keeping the water in the pump well at the lowest possible level at which the pump will take suction. The pump, which is of the centrifugal type, is located on the narrow neck of land south of the harbor, and is situated between the lake and the Milwaukee River. Plate II, which is a sectional view on an east and west line, shows the pump well, pump, engine and discharge into the lake. The average lift during the low-lake level for the past six years has been 15 ft., with an average discharge of about 39 000 galls. per minute.

This pumping engine, which was designed by Mr. Edwin Reynolds

and built by the E. P. Allis Company, of Milwaukee, has a capacity of 70 000 000 U. S. galls. per 24 hours, with a lift of 18 ft., and is a horizontal tandem compound engine with a 24-in. diameter high-pressure and a 43-in. diameter low-pressure cylinder and 48-in. stroke. The cylinders are steam-jacketed and fitted with Reynolds-Corliss valve gear. The main shaft of the engine is vertical, having the pump attached to the lower end, with the crank and fly-wheel moving in a horizontal plane. The pump disc is 12 ft. in diameter, with openings between the blades of sufficient size to pass readily ordinary-sized objects contained in sewage, and moves at a speed of from 65 to 70 revolutions per minute. The sewage is sucked from below and discharged from the rim of the pump disc through a circular casing into a 6-ft. diameter brick conduit, discharging over a weir into the lake. The suction well is provided with a gate, so that any part of the pump may be reached. Steam is supplied, at a pressure of 55 lbs., from three horizontal tubular boilers, each 60 ins. diameter by 16 ft. long, containing 54 4-in. tubes, with an additional boiler always in reserve.

This pump was started on August 1st, 1885, and has been operated continuously since, with the exception of a few weeks each spring, when it has undergone a thorough examination. The average annual duty developed by it has never been below 58 000 000 foot-pounds per 100 lbs. of coal consumed in the boiler furnaces, without any deductions of any kind, even for the heating or electric lighting.

The 56 000 000 galls. pumped each day, of which about 18% is sewage and refuse from packing-houses and other establishments in the valley which are all directly connected with this sewer, and the other 82% of which is river water, are discharged with considerable velocity out into the lake. With the exception of a very few days in the year, this discharge is carried off to the south with the lake current, which along the west shore of the lake is uniformly to the south, unless counteracted on the surface by a strong south wind or interfered with by a severe storm on the lake. The line of sewage current is well defined by its dark-brown color, which gradually, however, becomes lighter towards South Point, and at a distance of 6 miles from the works entirely disappears.

The result in the valley has been that, naturally, the discharge of sewage and offensive refuse into the canals and stream in the Menomonee valley, within the city limits, has ceased. This in itself, however,

did not improve matters in these canals, for although much dredging was done from year to year, yet the almost liquid sludge, which had accumulated for over 20 years, largely remained on the bottom, and merely required the raising of the temperature of the nearly stagnant water during the summer season to cause the development of gases, which, being absorbed by the water to saturation, and coloring it nearly black, then became perceptible and very offensive. By utilizing the excess capacity of the intercepting sewer in drawing water out of the upper ends of these canals, particularly since a supply of clearer cold lake water in the Milwaukee River through the flushing works has been brought to the mouth of the Menomonee valley, a gradual change is brought about. The water drawn out of each canal is replaced by cooler, clearer water, which checks the formation of gases, creates a slight current, and dilutes the sewage until the same becomes clear or nearly so ; and the canal waters that formerly were in hot weather a living mass of putrefaction became clear and clean enough to be used in steam boilers and to induce people to bathe therein. The excess capacity of the sewer, however, not being large enough to permit the drawing of the water from all the canals simultaneously, the connections are opened successively, as the conditions require. All the canals are thus kept in a very fair and good condition, excepting, however, the westerly end of the north canal, which receives the waters of the river from beyond the city limits, where they are polluted by the sewage and factory discharge of numerous institutions along its banks.

While this work in the Menomonee valley was being gradually extended as rapidly as the funds would permit, the Milwaukee River gradually became more and more offensive, until, in 1887, it was perhaps as black, foul and offensive as any stream in the country. As a relief, intercepting sewers along each bank of the river were generally demanded, notwithstanding the finances of the city and the constitutional bond limitation made it practically impossible to carry out such thorough and permanent work, excepting through a long number of years, and without any possibility of relief until the entire work was completed. In December, 1887, a commission of engineers, consisting of T. J. Whitman, J. P. Davis and Henry Flad, Members of the American Society of Civil Engineers, was appointed to assist in devising plans, with estimates of costs, for the construction of works for the

collection and final disposal of the sewage of the city and for the supply of a sufficient quantity of pure water for the city. This commission submitted its most complete and comprehensive report on May 21st, 1889.

Believing, however, that a speedy and prompt relief had become absolutely necessary, the writer recommended the construction of a large tunnel from the lake to the river at a point just below the dam, and the forcing of a volume of water through such tunnel sufficient to displace the entire volume of water in the river from the dam to its mouth each 24 hours, thus insuring a dilution of the sewage discharged into the river through 47 different sewers, sufficient to promote the natural process of purification, and to prevent the fouling of the waters or the formation of gases from the sludge on the bottom, and also to assist the work of improving the canals in the Menomonee valley. Notwithstanding a continuous and general opposition by almost every citizen and paper in the city, this work was finally authorized. It was contracted for December 8th, 1887, and completed sufficiently to allow the first flow of water to be forced through the tunnel into the river on the 14th day of September following, or within less than nine months after it was started.

The sewage and factory liquids emptied into the Milwaukee River were estimated at about 10 000 000 to 13 000 000 galls. daily, which, together with the gases continuously bubbling up from the river bottom during warm weather, did not take long each spring, and again within few days after each heavy rain, to make the sluggish stream a detestable nuisance. Finding, upon examination, a deposit of from 18 to 24 ins. of semi-liquid sludge all along the bottom of the river, and reproducing by experiment on a small scale the formation of gases from some of this sludge and their result upon the water, and ascertaining that a temperature of 69 to 71° was necessary to their development and that a dilution of the sewage with 25 to 30 times its volume of clear water was sufficient to prevent it from becoming noticeable, that proportion of clear, cold lake water to the sewage discharge was determined upon as the amount necessary to produce a speedy and prompt abatement of the river nuisance. The shortest distance from the lake shore to the river immediately below the dam being about 2 500 ft., and not desiring to produce a head of more than about 2 ft., the diameter of the tunnel was fixed at 12 ft., and the line located from

a point on the lake shore within and near the upper end of the harbor of refuge where the sand accretions had formed a beach, to a point within 200 ft. of the Milwaukee River just below the dam. From this point the tunnel was curved so as to approach a line more nearly parallel to the river, with the view of discharging the water into the stream at an angle that would not tend to retard unnecessarily the velocity of flow. The grade of the tunnel was made level and at such depth that the crown would be 2 ft. below city datum or supposed low water. At the lake end under the sand beach, another curve was made in the tunnel, to permit the placing of the pumping engine outside, but in direct line of the main tunnel, and to enable the taking of the supply from the harbor in a southerly and protected direction. A 12-ft. diameter wedge gate was placed in the tunnel each side of the pump chamber, and another having in its center a 3-ft. diameter auxiliary gate was placed near the outlet at the river, so as to permit the pumping out at any time of either the tunnel or pump chamber. The tunnel is a circle in section and encased with four rings or 18 ins. of brickwork. The outlet is constructed of stone through an offset in the dock having its face down stream. The inlet consists of two parallel piers made of a single row of piles 4 ft. apart between centers, backed on the outside with a double row of 3-in. pine sheath planking screw-bolted to a 12-in. cap timber fastened to the top of the piles. These piers extend out for a distance of 610 ft. from the beach into 10 ft. depth of water, and are placed 110 ft. apart, so as to reduce the velocity of the current between them, and toward the pump, sufficiently to prevent the moving of sand on the bottom of this channel, which was dredged to a uniform depth of 9 ft. Across the inner end of this channel near the beach line, a timber step resting upon piles and 2 ft. above the bottom of this channel was constructed as a further barrier against any moving sand entering the tunnel. This step also supports the 1-in. round iron bar grating across the channel, and forms the covered inlet proper, which is 65 ft. in width and converges in a distance of 26 ft. to a section of 14 ft. in width and 9 ft. in height. This section is retained for a distance of 130 ft., where it assumes a circular form and connects with the brick tunnel 10 ft. from the first gate. The rectangular conduit is constructed of timber and piling lined on all interior sides with 2 and 3-in. matched flooring. The screw pumping engine, which was designed by Mr. Edwin Reynolds to

meet the requirements of the specifications, was built by the E. P. Allis Company, and stands within a wrought-iron caisson 16½ x 26 ft. area, and 11½ ft. below city datum, on a concrete and solid stone foundation 8 ft. in depth, with the wheel shaft in direct line with the center of the main tunnel, driving the pump wheel at its end, as shown on the section on Plate III.

The engine is of the vertical steeple compound type with the high-pressure cylinder 19 ins., and the low pressure cylinder 38 ins. in diameter with a 48-in. stroke. The cylinders are steam-jacketed, the cylinder heads forming the valve chests, thus jacketing them with steam at the initial pressure for each cylinder. They are fitted with the Reynolds-Corliss valve gear with adjustable cut-offs for each cylinder. The shaft is provided with a water stuffing-box, and is fitted with a thrust-bearing and out-board bearing not unlike those of marine engines. The pump wheel is 14 ft. in diameter (the tunnel being enlarged at this point) with a cone-shaped hub 6 ft. in diameter. The wheel is a true screw and has a pitch of 8 ft. Beyond the wheel is the out-board bearing, this being supported by radial arms which are utilized as guide blades for converting the rotary motion imparted by the wheel to the water into a motion parallel to the axis of the tunnel.

Steam is supplied at a pressure of 100 lbs. by three horizontal tubular boilers, with one boiler in reserve. They are each 54 ins. in diameter by 16 ft. long, and contain 34 4-in. tubes. The engine and boilers are housed in a neat brick building. Annexed to the main building is a brick coal shed and tool room and a circular brick smoke-stack of 126 ft. in height, having an interior diameter of 4 ft. 4 ins. throughout.

The work on the tunnel was commenced on December 20th, and proceeded with fair speed at an average depth of about 95 ft. beneath the surface, partly under the center of a street and partly beneath private grounds and residences. It passed generally through a clay and hard-pan formation, with the exception of about 600 ft. in length where it penetrated a body of running sand and quicksand, and where progress was necessarily quite difficult and slow.

The work was divided into six contracts, viz.: the outlet, tunnel, inlet, pumping engine and boilers, foundations therefor and the buildings; and was prosecuted with such energy as to complete the entire

work, with the exception of the buildings, so as to permit its being operated within less than nine months from the day work was commenced. The tunnel was filled about noon of September 14th, and the machinery started. Up to the last moment the general opposition to the work had continued, and failure was predicted; but although the current in the river at the time of the starting of the pump happened to be slightly up stream, and its waters black, warm and foul, yet within 14 minutes the current in the river was reversed, and the clear, colder lake water seemed to push out the warm water in an almost solid body. Observations of the temperature of the water taken daily at a depth of 5 and 10 ft. below the surface and at the bottom of the river, at State Street, East Water Street and the Menomonee Bridges, from June 20th on through the season, showed that at East Water Street Bridge, the one nearest to the harbor, the water was from 1 to 1½° warmer at the 5-ft. depth than it was nearer the bottom, while at the other bridges there was no difference at any depth, and also that the water at the upper bridges was uniformly from 1½ to 2° warmer than it was at East Water Street. These observations were taken between 8 and 9.30 A. M. each day, and the average temperature at State Street Bridge for the month of June was 73.6°; for July, 73.4°; for August, 72.1°, and for the first half of September, 69.6°, the maximum being 76½° and the minimum 67°, the temperature of the water exceeding that of the air from 6 to 9° at that time of the morning, during the months above mentioned, in all but two instances. The color of the water was generally black and the odor very strong and offensive. Within 20 hours after starting the pump, the river assumed a color and transparency it had not known for many years, and the temperature of the water dropped on the following day to 60° and on the next days, respectively, to 58, 55, 52 and 47°, the average for the latter half of September being 53.9°. At the bridge across the Menomonee River a similar change was observed, excepting that the temperature was 2° higher, consequently the canals in the Menomonee valley were also being directly benefited, in having the supply of cold and clearer water brought at least 1 mile nearer, and without having first to overcome the natural flow from the Milwaukee River. Fermentation or the forming of gases along the river bottoms was at once checked and permanently stopped, excepting within a slip alongside of the upper Milwaukee River which received the discharge from two large sewers

and from a number of tanneries and large breweries, but which slip is since being flushed out by water drawn from the river above the dam. The condition of the river improved continuously until at times the water has been as clear, blue and transparent as in midlake, fish have again made their appearance, and fishing near the upper end has become a regular pastime.

The entire cost of this work, including right of way, etc., has not exceeded $223 000, or less than one year's interest of the cost of a complete intercepting sewerage system, while the entire annual operating expenses have never exceeded $25 000.

The result of this work has been most satisfactory in completely abating the Milwaukee River nuisance. The dilution of the sewage is so great that where the river water, discharged through its natural outlet into the lake, was formerly distinctly noticeable for many miles, and sharply defined from the lake water by its color and odor, now, excepting in times of the clay-colored flood water, no very great difference in color is perceptible. The same is true, although to a much less degree, with the effluent from the sewage pumping works, which, as heretofore stated, takes its course to the south. The new water works intake, now under construction, will, when completed, bring in the supply from a depth of 60 ft. in the lake at a point 1½ miles further out than the present intake, and still further remove all probability of even a chance contamination of the water supply from this source for a great many years to come, excepting as such may occur from some other source.

The pumping engine which has been and is performing its work with marvelous ease has never given even the least trouble. It runs continuously from some time in April, when the works are started, to the end of the year, when they are shut down for the winter, and develops constantly throughout the year a duty in excess of the 70 000 000 foot-pounds which were required by the specifications. Within six weeks after the pump was started, a series of tests was commenced to determine the volume of water which the pump was forcing through the tunnel.

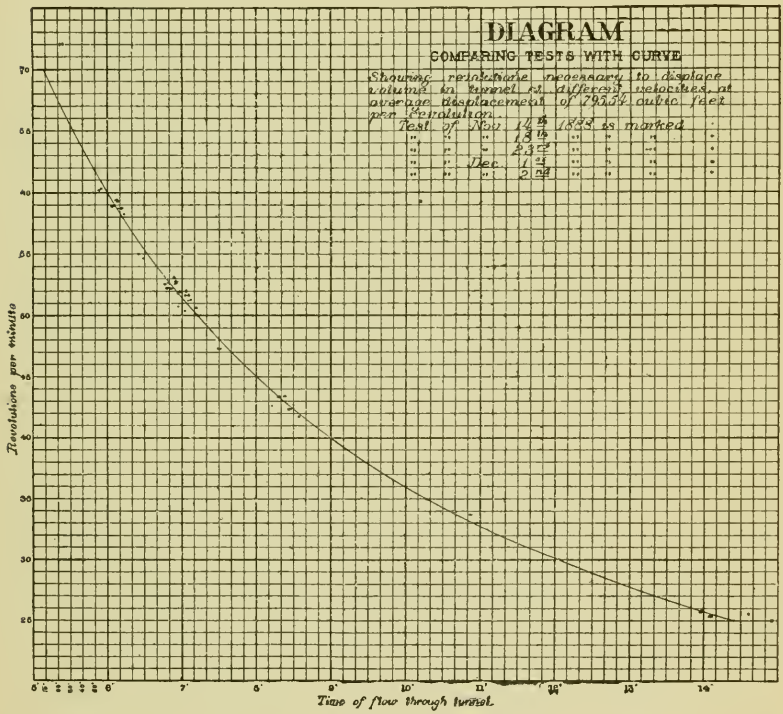
The outlet of the tunnel being entirely submerged, it became impossible to determine with any accuracy the velocity of flow in it by floats of any specific gravity. Either they would not become immediately visible at the outlet, or, if they did, they had been retarded

by contact with the arch, which facts became quite noticeable when a large number of different floats were used at one time. Eosine as a coloring matter was therefore used. There being a shaft over the center of the tunnel 5 ft. beyond the wheel, 2 ounces of red eosine, dissolved in 1 quart of water, was suddenly discharged at the center of the tunnel in this shaft and the time between its discharge and its appearance at the outlet noted, and the exact distance between the two points being known, the velocity was easily arrived at. The color was readily perceptible at the outlet, and was never distributed over a length of more than 7 to 9 ft., being about $\frac{1}{3}$ of 1% of the length of the tunnel, the center of which was taken as the point observed. The compactness of the coloring matter showed that the velocity was practically uniform at all points in the cross-section of the tunnel, which again in itself was very uniform throughout the entire length of the tunnel, as was shown from measurements taken at the time of its construction, immediately after its completion, and again in the following spring before the works were started, when the tunnel was emptied for the purpose of examination. The bricks which had been used for the interior were quite smooth, the joints were generally quite well pointed, and there were no obstructions to the flow, excepting a 4 x 6-in. block 18 ins. long, which had been overlooked and was found spiked to the brickwork at about the springing line. The first sets of observations were made with watches at each end, which had been carefully compared immediately before and after the tests, and by changing the observers so as to eliminate any difference due to personal error in observations, which in some cases was found to amount to from 2 to 5 seconds. The readings of the engine-counter were also noted immediately before and after each observation, in order to determine the average number of revolutions per minute during the time of each test, and also the total number of revolutions during such tests, or, in other words, the number of revolutions it required to displace the total volume of the tunnel between the points of observation. This total volume, being known, gave the average volume of water displaced per each revolution during such test. During the tests taken in this manner, which covered several days, the speed of the engine was varied for each double set of observations from 25 to 58.4 revolutions per minute.

In order to insure greater accuracy in these observations a telegraph

line with a key and bell at each end was run between the two points, and both observations of each test noted at each end of the line by stop watches, which were started the instant of the discharge of the eosine and stopped the instant of its appearance at the outlet. These observations of time taken at each end showed very little variation.

On December 1st and 2d, 1888, the 24-hour official duty and capacity test was made, during which time nine complete observations as to velocity of flow were made. There being another shaft over the center of the tunnel within 30 ft. of the outlet, the height of water in each of these shafts was very carefully noted at the beginning and close of each separate velocity test. The data of this trial run are hereto attached; also, the results of the velocity observations.



After the completion of the 24-hour run, during which time the speed of the engine did not at any time vary over 3%, the speed was increased to over 59, and then again reduced to about 25, revolutions

per minute, velocity tests being again made at the different intermediate speeds, the results clearly showing that each revolution of the wheel represents a displacement of 795.05 cu. ft., irrespective almost of what the speed or the head pumped against might be.

The fact is also clearly shown by the diagram, Fig. 2, showing in the curve the uniform average displacement of 795.54 cu .ft. for any speed from 25 to 60 revolutions per minute, and the observations of each day of the average revolutions and time of flow through the tunnel plotted thereon.

The data of December 1st and 2d are, perhaps, the more accurate, the observations having been made with the aid of the wire. To determine the total lift of the pump, the elevation of the water on each side of the wheel was carefully noted and found to have been 3.049 ft. The temperature of the water was uniformly 39°, and the weight at that temperature was assumed to be 62.424 lbs. per cubic foot. During the trial the total number of revolutions were 74 658, or an average of 51.8458 per minute, and the total amount of coal used was 14 750 lbs., hence the duty developed was 75 944 037 foot-pounds per 100 lbs. of coal consumed without deductions of any kind, and the volume displaced during the 24 hours equal to 58 854 395 cu. ft., or 440 220 867 U. S. galls., which is about the average daily work this engine has been performing during its regular run for nearly nine months in the year. At a speed of 59 revolutions per minute, this pump displaces 500 966 605 U. S. galls. per 24 hours, which is believed to be the largest quantity of water handled by any single machine in the world.

The above duty has since been exceeded, for, at the time the above test run was made, the engine and boilers had not been entirely enclosed by a permanent building. The average duty developed during last year's run was 74 654 440 foot-pounds, without any deductions for steam used for heating or for operating a 25-light electric dynamo. This also is believed to be the highest duty developed by any style of wheel pumping engine yet made.

Interesting, in connection with the data collected during and after the test run, is the fact that the value of the coefficient in the formula $v = c\sqrt{ri}$ for tunnel or conduits of 12 ft. diameter for velocities from 5 to 7 ft. per second is found to be from 122.7 to 128.3, with an average of 124.9 for these velocities.

DATA OF OFFICIAL TRIAL OF DECEMBER 1ST-2D, 1888.

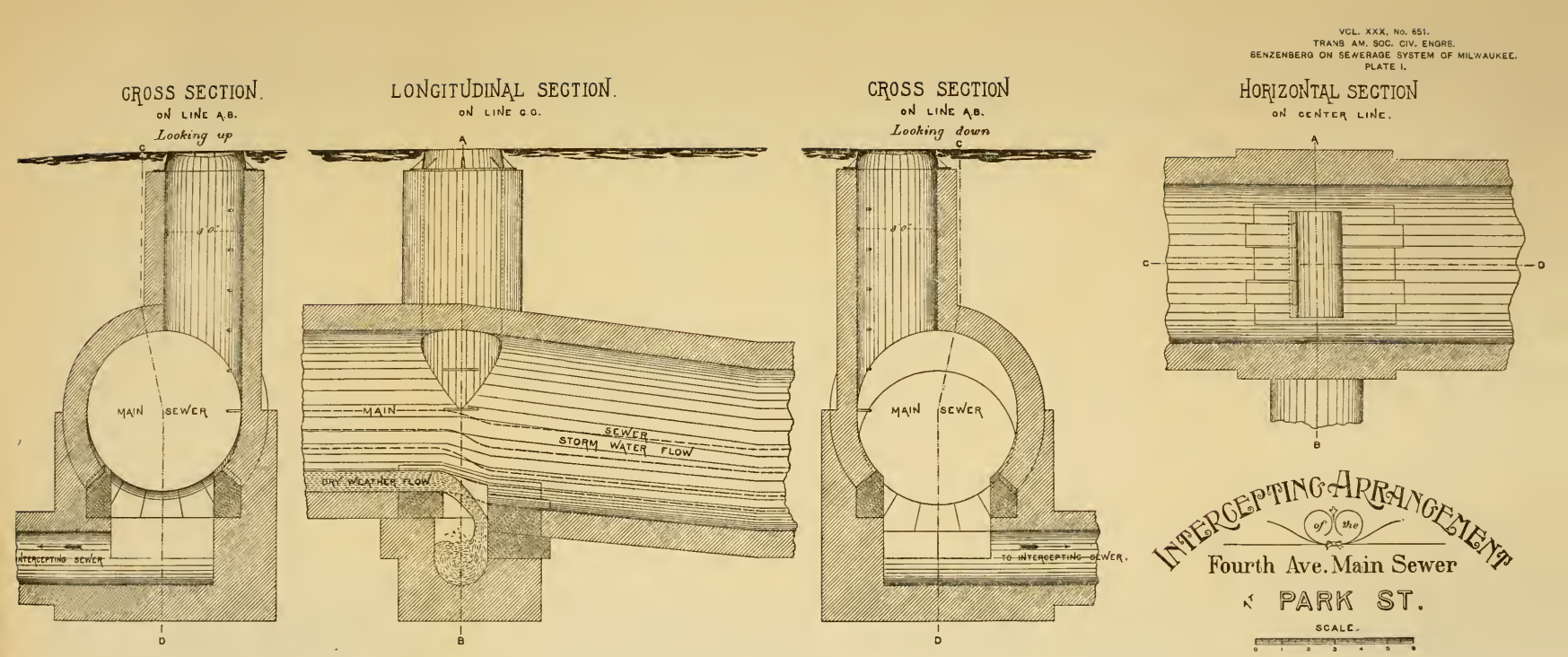
Duration of trial	24 hours.
Average steam pressure by gauge	102 lbs.
Average vacuum pressure by gauge	26 ins.
Total revolutions of engine	74 658
Average revolutions of engine per minute	51.8458
Piston speed, per minute	414.76 ft.
Average indicated H. P	325.47
Total coal (anthracite) fired to furnace	14 750 lbs.
Total ash and clinker	1 810 "
Square feet of grate area in use	56.25
Square feet of heating surface in use	2 025
Total water fed to boilers	109 890 lbs.
Temperature of feed water	120.62°
Temperature of water pumped	39°
Weight per cubic foot of water pumped	62.424 lbs.
Total lift of water pumped	3.049 ft.
Cubic feet of water pumped per revolution	788.32
Cubic feet of water pumped per minute	40 871.1
Cubic feet of water pumped during trial	58 854 395.
Duty developed, per 100 lbs. of coal actually fired	75 944 037.

DATA OF CAPACITY TESTS AND OBSERVATIONS.

Length of tunnel between points of observation	2 534 ft.
Length of tunnel between center of shafts	2 504 "
Sectional area of tunnel per foot	113.09 cu. ft.
Total capacity of tunnel between above points	286 570 "

Date of tests.	No. of tests.	Average number of revolutions per minute.	Observed time of flow through tunnel.	Total number of revolutions.	Actual velocity in feet per second.	Cubic feet water displaced per revolution.
1888.						
November 14.....	1	42.44	8′ 28″	359.32	4.988	797.53
"	2	42.60	8′ 12″	349.32	5.150	820 36
"	3	47.29	7′ 29″	353.89	5.640	809.77
"	4	47.29	7′ 31″	355.46	5.618	806.19
"	5	50.71	6′ 56″	351.59	6.091	815.07
"	6	50.33	7′ 02″	353.99	6.005	809.54
"	7	55.00	6′ 25″	352.92	6.582	812.00
"	8	54.60	6′ 29″	353.99	6.514	809.54
"	9	58.20	6′ 14″	362.78	6.775	789.93
"	10	58.60	6′ 8″	359.41	6.886	797.33
November 18.....	1	25 00	14′ 55″	372.92	2.831	768.45
"	2	25.61	14′ 34″	373.05	2.899	768.19
"	3	41.78	8′ 34″	357.92	4.930	800.65
"	4	42.44	8′ 27″	358.62	4.998	799.09
"	5	60.33	5′ 54″	355.95	7.158	805.09
"	6	60.16	5′ 53″	353.96	7.178	809.61
November 23.....	1	50.57	7′ 11″	363.26	5.879	788.88
"	2	51.17	7′ 5″	362.45	5.962	790.64
"	3	51.29	7′ 7″	365.01	5.934	785.10
"	4	lost.				
"	5	51.85	6′ 56″	359.49	6.091	797.16
"	6	52.00	7′ 3″	366.60	5.990	781.70
"	7	lost.				
"	8	51.62	7′ 2″	363.06	6.005	789.31
December 1.....	1	53.11	6′ 53″	365.57	6.136	783.90
"	2	52.00	6′ 58″	362.27	6.062	791.04
"	3	51.71	7′ 5½″	366.49	5.959	781.93
"	4	52.67	6′ 55½″	364.52	6.102	786.16
"	5	52.57	6′ 48¾″	356.38	6.229	804.11
"	6	52.80	6′ 55¾″	365.86	6.095	783.28
December 2.....	1	52.16	6′ 48½″	354.90	6.207	807.46
"	2	52.57	6′ 49″	358.35	6.195	799.70
"	3	52.14	6′ 51¾″	357.81	6.154	800.90
"	4	58.67	6′ 11½″	363.26	6.821	788.88
"	5	58.83	6′ 4″	356.92	6.961	802.90
"	6	59.17	6′ 8¾″	363.63	6.872	788.08
"	7	59.23	6′ 8″	363.27	6.886	788.86
"	8	43.41	8′ 22½″	363.56	5.043	788.23
"	9	43.30	8′ 18½″	359.75	5.083	796.58
"	10	33.73	10′ 52″	366.53	3.886	781.84
"	11	33.90	10′ 39″	361.03	3.965	793.75
"	12	25.38	14′ 5¾″	357.75	2.996	801.00
"	13	25.70	13′ 59″	359.37	3.020	797.42

VOL. XXX, No. 651.
TRANS AM. SOC. CIV. ENGRS.
BENZENBERG ON SEWERAGE SYSTEM OF MILWAUKEE.
PLATE I.



INTERCEPTING ARRANGEMENT of the Fourth Ave. Main Sewer AT PARK ST.

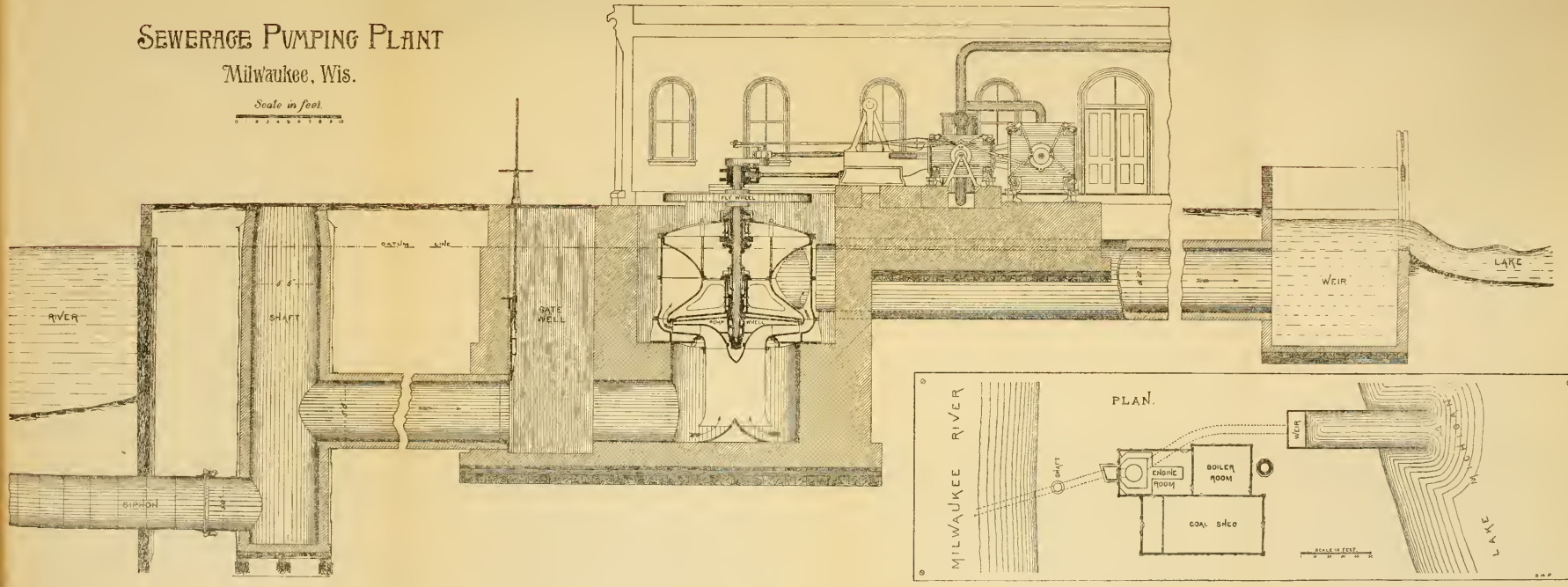
SCALE.

VOL. XXX, No. 651.
TRANS. AM. SOC. CIV. ENGRS.
BENZENBERG ON SEWERAGE SYSTEM OF MILWAUKEE.
PLATE II.

SEWERAGE PUMPING PLANT

Milwaukee, Wis.

Scale in feet.



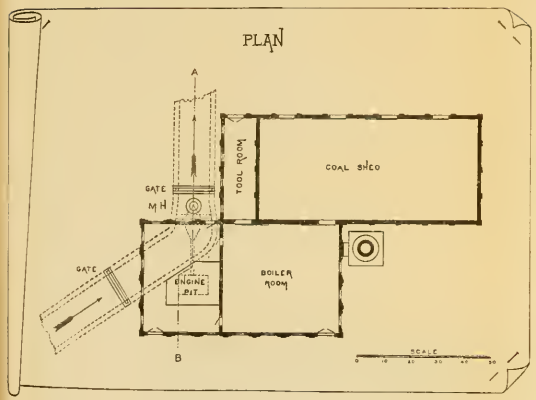
VOL. XXX, No. 651.
TRANS. AM. SOC. CIV. ENGRS.
BENZENBERG ON SEWERAGE SYSTEM OF MILWAUKEE.
PLATE III.

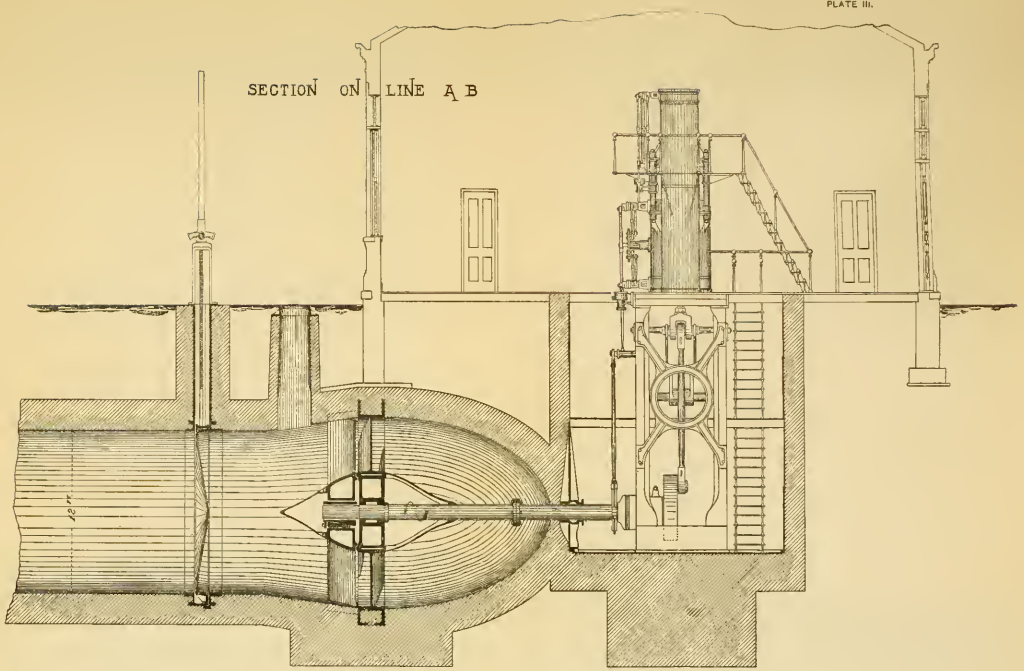
Milwaukee River Flushing Works.

PUMPING PLANT

SCALE,

0 1 2 3 4 5 6 7 8 9 10
FEET.





No. of test.	Observed time of flow through tunnel.	Actual velocity in feet per second.	Observed difference in water level in shafts 2 504 ft. apart.	Average observed head per 1 000 ft.	Coefficient (C) of formula $v = (c)\sqrt{ri}$
			Feet.		
4.....	6′ 11½″	6.821	2.48 2.58	1.010	124.02
5.....	6′ 4″	6.961	2.63 2.58	1.040	124.30
6.....	6′ 8¾″	6.872	2.62 2.62	1.046	122.71
7.....	6′ 8″	6.886	2.62	1.046	122.96
1.....	6′ 48¼″	6.207	2.09 1.88	0.793	127.19
2.....	6′ 49″	6.195	2.04 2.07	0.821	124.90
3.....	6′ 51¾″	6.154	2.11 1.97	0.814	124.57
8.....	8′ 22½″	5.043	1.31 1.40	0.541	125.13
9.....	8′ 18½″	5.083	1.35 1.27	0.523	128.35
10.....	10′ 52″	3.886	0.67	0.268	137.31

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

652.

(Vol. XXX.—November, 1893.)

THE ENLARGEMENT AND IMPROVEMENT OF THE NORTH SEA CANAL OF HOLLAND (AMSTERDAM SHIP CANAL).

By A. E. KEMPEES, C. E., Royal Engineer in Holland.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

The North Sea Canal of Holland.—The North Sea Canal (also known by the name of "Canal through the narrowest part of Holland") is the high road of commerce from the North Sea to Amsterdam, and is continued further eastward as far as the Zuyder Zee.

A description of the important works, which about twenty years ago were in course of construction in connection with the digging of this canal, has been given by Brevet Major-General J. G. Barnard, in the "Professional Papers of the Corps of Engineers of the United States

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

Army." Another account of the works executed before 1874 may be found in the well-known paper of M. Ph. Croisette Desnoyers, Ingénieur en Chef des Ponts et Chaussées, under the title, "Notice sur les Travaux Publics en Hollande."

The above-mentioned papers being known to the whole professional world, it may be judged superfluous to repeat what has been so correctly stated in them.

This paper is a sequel to those writings, and will endeavor to sketch in strict outlines what has been done since 1872 for the completion, maintenance and improvement of the North Sea Canal.

It is a well-known fact that the execution of the plan was undertaken by the Amsterdam Canal Company. This company found numerous technical as well as financial difficulties in its way. It is not to be denied, indeed, that on several occasions it met with the powerful support of the Government, but in the long run the means of carrying out the task failed, and on the 31st of October, 1881, an agreement was signed by which the company handed over all its possessions and rights, as well as its liabilities and obligations, to the State of the Netherlands. The Act of December 19th, 1882 (*Official Gazette*, No. 231) ratified this agreement.

Before this change in the direction of the works had taken place, they had been executed to such an extent as to have definitely established the new route of communication with the North Sea, to have completed the enclosure in the direction of the Zuyder Zee, and to have reclaimed and sold the several polders in the enclosed part of the Y.

In 1872, on the 10th of March, the locks on the Zuyder Zee side, near Schellingwoude, were opened for public service. They had previously, on the occasion of the laying of the memorial stone (April 29th, 1870) by His Majesty King William III, received the name of "Orange Locks."

In 1874 the sale of the first polder could be made; in 1882 all polders had passed into private hands, with the exception of a small strip of land (polder vii) close to the entrance of a side canal to Zaandam, which is, though surrounded by embankments, not drained. This enclosure is still being used as a deposit for material dredged from the canal.

The harbor on the North Sea having got a navigable channel 5.5

m. deep below Amsterdam water-mark (Amsterdam level, A. P.'*) and the canal itself a depth of 6.5 m. below A. P., the route was opened. A few days before the opening ceremony the necessary regulations had been established concerning the tariff of harbor and canal dues and the police regulations to be observed in passing the canal and the works belonging to it.†

A short time after the works had passed into the hands of the Government there was a general call for improvements of the various parts and for the extension and enlargement of the canal and its works. Considerable sums have already been spent for this purpose, extensive works are still being executed, but in order to make Amsterdam accessible to the largest sized vessels many more improvements will have to be carried out.

The present condition of the canal has been indicated in its chief outlines on Plate I. The most important of the new works in course of construction have likewise been represented upon it.

North Sea Harbor.—The outer harbor is bounded by two piers made of concrete, each 1 528 m. long, while the bases of these piers are 1 200 m. apart; their converging direction gradually diminishes this distance to 660 m. at 1 200 m. from the coast line. Then the piers curve inward so as to leave at a distance of about 1 400 m. from the coast line a harbor entrance 260 m. wide. Owing to the deposit of enormous loose blocks of concrete at the heads of the piers, a width of 200 m. remains for navigation.

The area enclosed by the piers (the outer harbor) comprises to the lowest line 92 hectares; to the high water-line, 102 hectares.

At one time an attempt was made to dredge in the outer harbor an elliptic basin having a width of 650 m. and a depth of 8½ m. below A. P. at the entrance, and of 7½ m. below A. P. on the shore side, but the work was stopped. From October, 1878, to November, 1879, 765 000 cu. m. of material, measured in the barges, were raised by dredging; but, notwithstanding this fact, the profile indicated that the material of the harbor had been diminished by but 4 000 cu. m.

At the present time by the aid of powerful dredges a channel of 250 m. width, 8.5 m. deep below A. P., is maintained; while, accord-

*A. P. means Amsterdamsch Peil, the average level adopted as a standard throughout the Netherlands by the King on the first of November, 1876, at which time the new harbor received the name of Ymuiden (mouth of the Y).

† Royal decrees of October 20th, 1876, *Official Gazette*, No. 27, and October 26th, 1876, *Official Gazette*, No. 189.

ing to the conditions of the present dredging contract, that channel will have to be deepened in 1894 to 9.50 m. below A. P. for a width of 250 m.

In order to perform this dredging, carried on also beyond the harbor mouth, and to which must be added the keeping at the requisite depth of the inner canal leading up to the North Sea locks, the following dredging plant must constantly be in good working condition at Ymuiden:

(*a*) Two hopper pump dredges of sufficient capacity to raise 455 and 610 cu. m. per hour of the matter found in and near the harbor mouth from a depth of 12 m. below the sea level.

The pump dredge, whose capacity is 610 cu. m. per hour, shall at the same time be fitted with ladder and buckets, to dredge in a draught of water of 11 m.

(*b*) Two stationary bucket ladder dredges of sufficient capacity each to raise and discharge into scows 270 cu. m. an hour of the material found in the harbor and in the outer canal.

One of these dredges must be able to do this work in a draught of 11 m., the other in a draught of 10 m.

(*c*) Five steam hopper barges, so-called "*ouderlossers*" (discharging dredgings through traps or valves in the bottom) with a capacity each of at least 150 cu. m.

Three of these steam hopper barges must be fitted with pump dredging machinery, two of sufficient capacity to raise at least 150 cu. m. of dredging matter per hour in a draught of water of 11 m. in and near the harbor mouth, the third to raise at least 285 cu. m. in that time.

(*d*) A large steam-tug of about 150 I. H. P., built and fitted up to serve as a direction steamer to the surveying staff.

(*e*) A sufficient number of reserve engine-parts, of which a full supply is constantly to be kept in readiness for immediate service in case of repairs being necessary.

The dredgings are discharged into the open sea at distances of at least 5 000 m. from the coast line wherever at a sea level corresponding to A. P. a draught of water of at least 13 m. is found. The quantity of material that has yearly to be removed in order to maintain the requisite depth, and in order to obtain the greater depth now desired, is enormous.

The following tables give an idea of the quantities raised since 1876. These quantities were not measured in place, but in the vessels used for transportation.

Year.	Outside the harbor's mouth. Cubic Meter.	In the harbor. Cubic Meter.	In the outer canal. Cubic Meter.
1876	.....	501 956	
1877	.....	936 804	
1878	53 118	1 642 098	19 009
1879	83 752	438 764	43 050
1880	47 424	264 085	27 920
1881	63 381	394 359	60 875
1882	86 413	390 124	37 710
1883	50 538	387 485	62 894
1884	69 775	504 122	84 875
1885	45 819	375 902	52 263
1886	83 257	550 829	36 035
1887	75 155	401 391	22 120
1888	56 370	376 055	30 900
1889	59 605	364 200	14 485
1890	82 848	368 917	18 710
1891	61 325	402 575	20 300
1892	87 905	751 455	39 120

During a considerable part of the year, the surf and swell made dredging impossible. The number of days when dredging could not be done amounted to—

Year.	Outside the harbor mouth.	In the harbor.	Year.	Outside the harbor mouth.	In the harbor.
1883	240	156	1888	270	136
1884	263	193	1889	227	103
1885	298	204	1890	207	119
1886	241	165	1891	255	126
1887	265	130			

The piers protecting the harbor against the North Sea were finished for their entire length in 1877.

During the construction of these works on a sandy beach exposed to the violent attacks of the North Sea, many difficulties and disappointments had to be faced.

From the outset it had been the intention to construct these piers of heavy concrete blocks. The dimensions, as well as the formation of these blocks, have often during the progress of the work been altered, while the cross-section of the piers appeared to be in urgent need of enlargement.

On Plate II the different modes of construction have been given in detail.

The original plan of the piers was based upon the system adopted at many English ports, where on a rocky foundation soil, from which the upper layer of clay or sand has been removed, a massive wall with almost perpendicular sides is erected without any protection of the foot against abrasion being necessary.

The inefficiency of similar constructions on the Dutch coast soon manifested itself in an unmistakable manner. Figs. 1 and 5 of Plate II, showing cross-sections of the North Pier as it was first made, remove all doubt on this subject.

At that period only the side fronts of the pier were constructed of concrete blocks prepared in advance; between those walls a nucleus of sand originally, of concrete afterwards, was inserted.

After the winter of 1867, when some experience had already been gained, the pier was constructed entirely of blocks of concrete prepared in advance, the bottom layer of which had imbedded itself into the sand and had afterwards been trimmed to a horizontal plane by the divers (Fig. 2).

This way of proceeding was relinquished for the one sketched in Figs. 3 and 4, which gave a solid basis to the work by the application of a rubble of basaltic blocks forming a stratum 1 m. high between lines of concrete blocks (Fig. 3), and protruding as far as 10 m. beyond the foot of the piers.

Notwithstanding these alterations, the end had by no means been gained. Figs. 8 and 9 certainly do not, in giving a faithful representation of the condition of the north pier at 700 m. from the coast line, convey any notion of a work in every respect a match for a sea that can so mercilessly lash the Dutch coast.

In 1873 a commission of experts was appointed by Royal decree for the purpose of investigating some questions relating to the North Sea Canal.

One of these questions was : "Are the North Sea piers to be judged of sufficient capacity, both as regards construction and strength of materials, and, if not, what are the improvements required ?"

Up to that time the concrete blocks for the side facings of the piers had been composed of 1 part of Portland cement to 4 parts of sand (from the sand hills) and 5 parts of gravel.

The blocks were 1.067 m. (3 ft. 6 ins.) high, 1.219 m. (4 ft.) broad, and 1.524 m. (5 ft.), 2.134 m. (7 ft.) and 2.743 m. (9 ft.) long.

The Commission arrived at the following conclusions:

The nethermost blocks do not form a fixed basis, and the upper blocks are stated to shift places to a considerable degree.

The basis has not, at least from the sea side, been sufficiently secured against abrasion.

The Commission was further of opinion that the formation of the mortar used for the blocks, and also the dimensions of the same, were insufficient.

Accordingly it proposed:

First.—To start from the principle, in securing the basis, of the protection having to be carried on to the greatest depth found alongside the work.

And, although it cannot *a priori* be made out what breadth will be sufficient, the Commission feels justified in assuming that the basis to be constructed on the bottom of the sea should protrude at least 10 m. on that side of the piers facing the sea beyond the foot of the rubble to be mentioned below, as the breakwater will be deposited on that basis.

The bottom layer is to be carried on round about the heads of the piers for the same breadth and to be continued as far as will appear necessary along that side of the piers facing the harbor.

As to the remaining part of the harbor front, wherever, during the progress of the work, abrasion takes place, the basis to be deposited is also to extend to at least 10 m. beyond the foot of the pier. The height of the bottom layer may be fixed at 1 m.

Second.—To adhere, in protecting the pier from the impetus of the waves, to the principle of depositing at the seaside rubble reaching from the basis to the top of the pier, and thus forming a breakwater in front of it.

The core of this protection may be composed of stone or small concrete blocks; the outer facings of the breakwater should be made of heavy blocks of concrete.

Inasmuch as, in the absence of examples of similar works on the Dutch coast, the Commission dare lay down an opinion, it ventures to propose the following dimensions to be given to the breakwater:

The nucleus of stone or smaller blocks of concrete may reach to 1.5 m. below the low water-line, with a top breadth at that level of no more than 3 m., and an outward slope 1.5 to 1. The facing

of heavy blocks covering the nucleus is to reach to the top of the piers, *i.e.*, to 4.10 m. above A. P., and to have a top breadth of at least 3 m. by an outward slope no steeper that 1 to 1.

The minimum weight of the blocks to be used for that purpose is to be fixed at 20 tons.

The rubble (breakwater) is to commence, for the north pier at 650 m., for the south pier at 750 m., from the coast line, go round the heads of the piers, and to be continued inward as far as will prove necessary.

The Commission does not judge it proper to give definite specifications for the formation of the concrete, but wishes merely to observe that the mixture hitherto used is of too weak a nature; the proportion of Portland cement ought to be increased, and sharp sand should be used instead of sand from the beach.

As for the piers themselves, it is observed that the parapet should be done away with on the north pier from 650 m. and on the south pier from 750 m. from the coast line, and that the piers should be fully raised to the level to which the top of the parapet now reaches. The top part, to be raised to the level of the actual parapet, should be one homogeneous mass across the whole section. A weight of 10 tons per block is judged necessary for the piers above low water.

In consequence of these remarks, the piers were given the cross-section traced on Plate II, Fig. 6. The blocks of concrete were composed of 1 part of Portland cement, 3 parts of sharp sand and 5 parts of gravel. Experience has taught that blocks of this formation can stand the attacks of the waves and the weather better than those previously used. The piers, as well as the breakwaters, continually require great sums for repairs.

In 1881, by way of experiment, the crown of the north pier was coated with masonry of hard-burnt bricks in strong mortar of Portland cement. This experiment was a complete success, and at the present moment both piers have this protection along their entire length. Fig. 7, Plate II, may now be considered as the type of cross-section of the piers.

The present formation of the concrete blocks for the breakwaters is 2 measures of Portland cement, 3 measures of sharp sand and 5 measures of clear gravel. The largest blocks are 3 m. long, 1.5 m. broad and 2 m. high. Their weight is about 19 500 kg. (20 tons).

On Plate III the present condition of Ymuiden Harbor has been reproduced.

The direction of the axis of the harbor is indicated by means of two white, fixed, dioptric lights of the first order. The western (or outer) light reaches to 41.6 m. above ordinary high water, and is visible at 18 nautical miles; the inner or eastern one reaches 51.5 m. above high water, and is visible at 19 miles. The lights are 560 m. apart and placed in round iron towers, painted with broad white bands.

On the heads of the piers are fixed lights of the sixth order, reaching to 12.7 m. above high water, and are visible at two miles. The north pier light is red at the seaside and green at the harbor side; the south pier light is green at the seaside and red at the harbor side. The lamps have been constructed so as to keep alight several days and nights running whenever the piers are inaccessible on account of bad weather.

Between the piers the navigable channel is indicated by five black buoys on the north side and five white ones on the south side. During dense fogs a steam siren whistle is worked, placed near the western light tower.

Since the opening of the canal a semaphore or coast signaling station has been erected at Ymuiden in connection with the telegraph service. This semaphore has been placed on the south side, near the outer light tower. In accordance with the "Universal Code of Signals," messages can be sent to and received from ships.

The harbor number of Ymuiden is 3. The average difference between tide and ebb is 1.6 m. at neap tide and 1.8 m. at spring tide. Water signals are hoisted at the yardarm of the semaphore's signaling mast in order to indicate every change in the draught of water. These signals, by night and by day, are given on Plate IV.

A few hundred yards inland, between the sandhills north of the harbor, is a beacon, placed so that the azimuth of the line (compass beacon—light on the head of the south mole) is S. 77° 34′ 24″ W. Two lifeboat stations are in close proximity to the harbor.

Sailors will find exhaustive information on the hydrographic chart of Ymuiden and in the accompanying description, published by order of the Naval Office, and in Halverhout & Zwast's "Guide to the Harbor of Ymuiden and the North Sea Canal."

The harbor between the piers cannot, in stormy weather, be consid-

ered a harbor of refuge, in the proper sense of the word. Experience has taught that, generally speaking, it may be entered by ships without any danger at all. During gales, when the passage through the locks was impossible, necessity has sometimes turned it into a harbor of refuge, not unsuccessfully. As a rule, however, vessels, after entering the harbor, should make for the outer canal with the least possible delay, to pass on to the canal through the North Sea locks.

It is worth mentioning that even in the exceptionally severe winter of 1890 and 1891 the traffic in the harbor of Ymuiden was only suspended for a very few days. The navigation in the North Sea Canal itself was in that winter attended with extraordinary difficulties, and was for a considerable time altogether suspended. Measures are planned to prevent similar suspension of traffic in the future by the aid of powerful ice-breakers.

The outer canal or inner harbor enters into the outer harbor between two stone moles on the heads of which fixed white harbor lights have been placed. These lights reach 5.3 m. above high water, and are visible at two miles. The outer canal is 1 430 m. long; it is kept at a depth of 8.5 m. below A. P. This depth is to be found in a channel gradually narrowing from 109 to 38 m. near the North Sea locks.

In virtue of the conditions of the present maintenance contract, this canal will have to be deepened to 9.5 m. below A. P. on July 1st, 1894. The bottom width will, at that time, be 85 m. at the entrance, and 35 m. near the locks. The banks of the outer canal are protected below A. P. by osier work, and above that level by stone sloping.

An enlargement of the outer canal, at the north side, now serves as a mooring place for numerous fishing craft, for whose convenience five jetties have, moreover, been built at the south side. With a view to the great development of the Ymuiden fish trade an area of 17 hectares has been purchased at the south side of the outer canal where a fisheries harbor will be dug.

The plans for the construction of that harbor are still preparing, but meanwhile large quantities of sand have been removed from the purchased grounds. This sand, as will afterwards be seen, is utilized in constructing a bar in the open Y, near Schellingwoude.

The outer canal, as yet, only leads to the set of locks already described on the twenty-eighth and following pages of the before-mentioned paper by Brevet Major-General Barnard.

The principal dimensions of these locks are:

Great tide lock, containing three pairs of tide gates, and two pairs of ebb gates, width, 18 m.; depth, 7.9 m. below A. P.; length of the combined lock pond, 120 m. The lock chamber is divided into two sections by a pair of tide gates. These sections are 70 and 50 m. long.

Small tide lock containing three pairs of tide gates and two pairs of ebb gates, width, 12 m.; depth below A. P., 5.14 m.; length of the lock pond, 70 m.

Outlet sluice containing two pairs of tide gates and one pair of ebb gates, width, 10 m.; depth below A. P., 5.14 m. The outer tide gates reach to 5 m. above A. P.; the highest storm-tide known at Ymuiden, on December 12th, 1883, ran up to 3.30 m. above A. P., near the locks.

Some years after the opening of the locks to trade, the conviction was arrived at that their dimensions were too small for large-sized craft.

As far as their length was concerned, this difficulty could be evaded either by locking the big vessels between the outermost tide gates and the innermost ebb gates (giving an increase of length of 8 m.), or by allowing them to pass at the moment when the tide, being on a level with the water in the canal, did not require any of the gates to be shut. Both these expedients offer peculiar difficulties and impede the regular course of traffic. But, as far as width and depth were concerned, nothing but the construction of new locks could remove these obstacles.

As early as 1885, a plan was sketched in outline for the building of a second great tide lock. According to this sketch the new lock was to have obtained a width of 25 m. at the level of the canal, and a depth of 8.5 m. below A. P. The length of the lock chamber was to have been 205 m., while a pair of intermediate gates was to divide it into two sections about 75 and 130 m. long. The outer front was to have retreated inland about 400 m. from that of the existing locks, the longitudinal axis to have been fixed about 180 m. north of the axis of the present canal. New branch canals would have had to connect these locks with the inner and outer canals.

On August 28th, 1886, a project for a law was offered to the Second Chamber of the States General authorizing the appropriations necessary for the enlargement of the harbor and locks of Ymuiden.

This project embraced the construction of the aforesaid fisheries harbor, as well as the building of a new lock of the above capacity. The cost of these works were then estimated at 4 246 900 guilders. The bill authorizing the appropriation was passed and published in the *Official Gazette*, No. 98, of May 31st, 1887. Very soon afterwards the elaboration of the plan for the lock was proceeded with. The necessary inquiries into the nature of the grounds, including five borings, were made. These borings showed that the soil was almost exclusively composed of sand or sand mingled with shells. Only here and there at different depths vestiges of clay were met with. At a depth between 17 and 19 m. below A. P., a substratum of solid loam was found supported by very solid and dry peat. This foundation soil might, therefore, be called of excellent stability, and the use of piles, unavoidable in the construction of most important works on the Dutch soil, is here superfluous.

There were fears, however, that the sand reaching down to so enormous a depth would throw almost insurmountable difficulties in the way of the execution of the work. The proximity of the present canal, which would be separated from the excavation by an embankment of only about 60 m., and that the top, justified the fear of a great strain of water from which inroads of sand might be expected. These considerations led to the conclusion that the foundations of the lock walls would have to be made under water by means of compressed air, and the walls and floors would have to be constructed thereon between iron fencings. The nether part of the side walls of the locks would have to be constructed of piled-up blocks of concrete.

During the elaboration of this project of foundations by means of compressed air, the digging and dredging necessary for these foundations were separately contracted for, together with part of the inner junction canal (for 348 500 guilders), and also the construction of 6 030 cu. m. of concrete blocks for the side walls (for 96 900 guilders). Permission was moreover granted to several private people to carry off sand from the grounds, while another portion was turned to account in constructing military works on the Amsterdam line of defense.

On October 30th, 1888, the interior digging for the locks commenced; for the most part excavators were used, the material being transported by rail to the beach north of Ymuiden. Although it had been taken for granted that in making the excavation a great

quantity of the matter would have had to be raised by dredging, it appeared during the progress of the work that even at a depth of 11 m. no leak was to be discovered. The water filtering through the adjacent downs, and through the embankment between the excavation and the North Sea Canal, could easily be pumped out by a few centrifugal pumps. In the days of the greatest influx of water the quantity thus removed never amounted to more than 8 000 cu. m. per 24 hours.

Towards the close of the year 1889, the plan of the lock built on foundations by means of compressed air was finished, and on the 27th of December it was to have been contracted for. In elaborating the plan some principal dimensions had been a little increased. The outermost mitre-sill was to have reached to 9 m. below A. P. The intermediate and innermost sills to 9.30 m. below A. P. The total length of lockage had been increased to 225 m., while the width had been kept at 25 m.

In the period intervening between the passing of the appropriation bill and the completion of the plans, the principal dimensions to be given to the locks had formed a subject of controversy in the daily papers and in various technical periodicals. Many professional men backed the opinion that, for the depth, the figure adopted by the Government ought to be increased. The subject was also discussed in the Second Chamber of the States General, and on December 20th, 1889, a considerable majority voted for the mitre-sills being lowered to at least 10 m. below A. P. Accordingly no contract could be made for the construction of the lock and the plan had to be recast.

As a result of the experience gained meanwhile with respect to any danger of inundation threatening the excavation, it was resolved not to put in foundations by means of compressed air and to build the locks on a basis of concrete.

The contractors of the interior diggings agreed to make the concrete foundations of the lock walls for 536 750 guilders. According to the conditions of the agreements drawn up for this purpose, grooved sheet piling has been applied around the four sides of each front lock wall and in front and at the back of each side wall. The sheet piles are 0.15 to 0.20 m. broad, 8 m. long, and their points reach to 16 m. below A. P. The bottom of the excavation had previously been deepened to 8.50 m. below A. P. wherever this piling had to be driven in, this driving having been further facilitated by squirting

in water from two sides. More than 2 000 cu. m. of fir wood were used for the formation of these sheets.

Each front wall and each side wall having thus been confined to a separate section, these sections were deepened to 13.70 m. below A. P. by means of a bucket-ladder dredge and a suction pipe, the level of the water during this work being kept at 9 m. below A. P. As soon as a section had reached the required depth, the filling up with concrete was proceeded with.

The concrete was deposited under water through wooden tunnels and in two strata of a thickness of 1.25 m. each. In the front walls these tunnels were moved across a floating scaffolding; for the side walls across braced beams, the tunnels corresponding to the two layers succeeding each other as closely as possible.

In four months, by dint of working night and day, the total bulk of 25 000 cu. m. of concrete had been cast in, the greatest quantity disposed of in 24 hours being 400 and odd cubic meters.

The silt proceeding from the concrete had been removed as much as possible during the execution. On the layer of concrete a layer of sand 2 m. thick was spread, beneath which the whole mass could harden undisturbed during the winter of 1891 and 1892. The concrete was composed of 6 parts of hydraulic lime (Tournay stone lime) measured after the slacking; 5 parts of sharp sand, 7 parts of trass and 17 parts of broken bricks.

These broken bricks had to be for at least two-thirds hard-burned clay, the other third part might be one degree less hard; they might not be larger than 75 nor smaller than 5 cu. cm. The contractors were at liberty to use, instead of hard-burned clay rubbish of Liege limestone, granite or any other hard and granulous stone.

In March, 1892, the removal of the water and the sand overlying the concrete in the different sections was commenced, and it appeared that, with the exception of three places, where springs had creviced the concrete, the foundations were a great success. The upper stratum of the concrete consisted, to a depth of 10 or 20 cm., of loose bricks and sharp sand of some coherency; the underlying concrete had become perfectly hard. The layer of inferior quality was removed with the pickaxe, and in its stead a thin layer of concrete of Portland cement was deposited, on which the masonry of the walls has been erected.

The sheet piling proved to be very tightly closed, in many places thin jets of water indeed squirted through the seams, but it was a very easy matter to remove this water together with that proceeding from the three springs by the help of two centrifugal pumps of a suction-pipe of 20 cm. diameter, so that the surface of the concrete at a level of 11.20 m. below A. P. was laid, comparatively speaking, dry. The site of the springs corresponds pretty closely to the spots where the borings were performed. Their water is fresh and impregnated with few gases, so that it evidently proceeds from underneath the stratum of peat at a depth of 19 m. below A. P.

The general system originally adopted for the repression of these springs was the removal of the bad concrete and the filling up of the hole with strong concrete of Portland cement, an iron tube fitted with a very large circular flange at the bottom having first been stuck in. Through this tube the spring could discharge itself without causing any inconvenience. These tubes might be filled up when the lock was getting finished.

The spring in the foundations of the outermost front wall has been very successfully treated in this way. The tube over this spring discharges 270 cubic dcm. of water per minute. Before the above-mentioned measures were taken, part of the water from the springs found its way alongside the western sheet-piling, but this leak has stopped of its own accord. Fortunately, these bad spots do not at all interfere with any part of the lock which has to resist the pressure of the water.

As a temporary measure, intending to counterbalance the upward pressure by placing a column of water over the damaged parts of the concrete, the spring in the foundations of the outermost front wall has been surrounded by a circular wall, which might now be broken down and thrown in the intermediate foundation by a wooden coffer.

This spring in the foundation of the intermediate lock piers pours forth about 1.2 cu. m. of water per minute. It has been decided to repress these springs by means of compressed air. This process will lay dry the damaged concrete, which will be removed and supplanted by more reliable matter, while the spring itself will again be led off through a tube until the completion of the work. Then the opening may be filled with Portland cement mortar or closed by any other device.

In the beginning of 1891 the plan of the superstructure of the locks had already so far been completed as to allow a contract to be made. This work comprises the building of the three sets of lock piers or front walls, of wings to the outer and inner front walls, and of the side lock walls ; the construction of fascine works outside the locks in front of the gates, and of the paving of the bottom of the lock chamber, besides the digging of branch canals on either extremity of the locks, inclusive of osier work, stone slopings and some works of minor importance.

The cost of the work under this contract amounts to 1 634 964 guilders. It is now under construction.

Plate V gives an idea of this colossal enterprise.

The principal dimensions are now :

Length of the entire lock, 282.16 m. ; length of the lock pond between the outer point of the outermost mitre-sill and the abutment of the outer gate recesses of the intermediate lock piers, 70.445 m. ; length of the lock pond between the point of the outer mitre-sill of the intermediate lock piers and the abutment of the outer gate recesses of the innermost lock piers, 144.445 m. ; length of the lock chamber between the point of the outer mitre-sill of the outermost lock piers and the abutment of the outer gate recess of the innermost lock piers, 225.775 m. ; width at a level of 3 m. below A. P. and lower down, 25 m. ; top of the mitre-sills, 10 m. below A. P. ; top of the copings of the front of the outermost lock piers, 5 m. above A. P.; top of the copings of the rest, 3.60 m. above A. P.

A number of other dimensions have been indicated on the plate.

The whole building is constructed of brickwork, with limestone facings and granite copings wherever necessary. The lock walls get a front facing of rough basalt down to 3 m. below A. P. The floor of the lock chamber will be paved with blocks of concrete, for which purpose the before-mentioned blocks, originally intended for the construction of the side lock walls under water, will be used. The construction of this lock shows few deviations from the system hitherto followed in the Netherlands. Separate mention should be made of the arrangement to fill and empty the lock, which is as yet only the second of that kind in the Netherlands. On either side, running the whole length of the side walls, have been constructed sluiceways 2 m. wide and 3 m. high, communicating with the lock basin by 11

openings, and closed by double sluice gates or paddles in each of the lock piers. Owing to this arrangement the filling of the lock basin will not, under the most untoward circumstances, take more than 10 minutes, while the inflowing or outflowing currents are distributed so as to cause none but the slightest inconvenience to vessels. Excepting the before-mentioned springs in the concrete foundations, making particular measures peremptory, no accident has delayed the execution of the work up to this date.

The plan of the iron gates is ready, and they, too, will soon be under construction. The way in which the gates and paddles will be worked has not yet been decided upon. The system will be either hydraulic or electric. The whole work will, to all appearances, be completed in 1895.

Starting inland from the North Sea locks, the canal cuts its way through a stretch of sandhills, about 4 700 m. long, to run on through the now drained Wykermeer and through the Y, the waste of waters changed into arable land since the execution of the work.

Due provision has been made for the requirements of the navigation and drainage of the adjacent countries by the construction of some side canals.

These side cuts are the following:

A. To Beverwyk.

B and *C.* To Spaarndam, a place very important, for the drainage of Rhynland (the Rhine polders), where at the same time a set of locks connects the canal with the water-way to Haarlem.

D. Towards Nauerna, where an outlet sluice for the Schermer polder is situated, and at the same time locks give access to that polder.

E. Towards Westzaan, where locks give access to the Westzaan polder.

F. To Halfweg (another important drainage station of the Rhine polders).

G. To Zaandam, a city whose commercial prosperity has greatly increased since the completion of the North Sea Canal. At Zaandam the side canal terminates into three locks partly used for navigation, but at the same time acting as outlet sluices to the Schermer polders.

H. Towards the mill sluice at Barndegat, an outlet for the Oostzaan polder.

I. Towards Oostzaan, where a lock gives access to the polder of that name.

K. Towards Nieuwendam; here a lock gives access to the Waterland polder.

These branch canals may be entered by vessels of the following draught:

For the canals	*B, C, D, F*	3.00 m.
" canal	*K*	2.80 "
" "	*E*	2.20 "
" "	*I*	1.80 "
" "	*A*	1.50 "
" "	*H*	1.30 "
" "	*G*	7.10 to 7.30 "

The following table gives some details about the polders reclaimed during the construction of the canal:

Name of Polder.	Area. (Hectares.)	Summer level. Meters below A. P.	Drainage Machines. (All worked by steam.)
Noord en Zuid Wykermeer	482	2.40	1 Centrifugal pump, 12 H. P.
" " Spaarndammer polder	520	2.60	1 " " 12 "
Houtrak polder	1 350	3.00	2 pumps, 80 H. P. (together).
Nauernasche, Westzaaner en Zaandammer polder	612	3.40	1 Centrifugal pump, 20 H. P.
Groote Y polder	1 950	3.10	1 " " 64 "
Amsterdammer polder	400	3.40	1 " " 19 "
Noorder polder	465	3.20	1 " " 20 "

The polders are protected by embankments whose crowns are nearly everywhere at a level of 1 m. above A. P. The breadth is, at the rest, 4 or 5 m. as a rule; the slopes on either side have 4 base to 1 rise.

Within the range formerly occupied by the Y, there is another polder called Benks loter Ham, covering 224 hectares, having a summer level of 2.23 m. below A. P. and for drainage purposes a centrifugal pump of 16 H. P. This polder had been reclaimed before the construction of the North Sea Canal, and as at that time the water of the Zuyder Zee had free access to the Y, it had been surrounded by an embankment much higher than those of the polders afterwards reclaimed. The crown of the dike is 3 to 3.50 m. above A. P.

The market value of the Y territories amounted to more than 2 000 guilders per hectare. On the whole these polders have a very fertile soil.

The North Sea Canal runs along these polders in a succession of long straight sections connected by weak curves, and cuts through only one polder (near Bentenhuizen) which existed before the construction of the canal.

The canal is bridged over at three points; near Velsen there is a swingbridge for ordinary traffic and another for railway traffic; and near Zaandam there is a swingbridge for the railroad from Amsterdam to den Velder.

The railway bridge at Velsen offers a passage 19.20 m. wide, the bridge for ordinary traffic one 19.25 m. wide; the railway bridge near Zaandam two, 19.50 and 19.15 m. wide.

The banks of the canal are further connected by two ferries, at Bentenhuizen, by one near Zaandam and by a steam cable ferry at Amsterdam.

On Plate VI has been drawn, in the first place, the cross profile of the canal at the outset. With regard to the depth it should be observed that the drainage interests of the surrounding country made a level of $\frac{1}{2}$ m. below A. P. desirable, a level which every effort is made to maintain as best may. The drainage and outlet arrangements now extant have to this date proved inadequate to perform this task without interruption, and a higher level is accordingly not unfrequent.

The conditions of the concession (charter) granted to the Amsterdam Canal Company stipulated a depth of 7 m. below the canal level and a bottom width of 27 m., the slopes on either side not to be steeper than 2 base to 1 rise. At the opening of the North Sea locks for traffic, neither the main canal nor the side cuts were found to contain the requisite draught of water. Vessels drawing more than 5 m. could, for the time being, not be admitted to the main canal. Gradually this state of things changed for the better, the profile of the canal was not only deepened, but also enlarged at several points. Even before the opening of the canal the City of Amsterdam had entered into an agreement respecting this enlarging of the canal and this increase of depth to 8.2 m. below A. P., or to 7.7 m. draught.

The task was performed between the years 1877 and 1883, while, in addition, a widening of the canal in the cutting through Bentenhuizen was brought about in 1887. In this way the canal obtained the profile represented in Figs. 2, 3 and 4 of Plate VI.

The present Special Regulations of Police for the North Sea Canal

(established by Royal Decree of May 23d, 1892, *Official Gazette*, 113) allows to vessels entering the canal a maximum of draught, wherever measured, of—

7.30 m.	at a level	of	0.30 m.	below	A. P.	and higher.
7.20 "	"	"	0.40	"	"	"
7.10 "	"	"	0.50	"	"	"

With a view to the draught of inward-bound vessels being increased through their passing from sea water into fresh or brackish water, the regulations direct that for the draught such vessels will get in the canal a standard of 0.1 m. shall be added to their draught in the outer harbor or outer canal. The maximum of breadth allowed to vessels is now 17.75 m. for the main canal; the maximum of length, 115 m., for lockage through the North Sea locks, and 90 m. for lockage through the Orange locks at Schellingwoude. The present dimensions of the canal and harbor are by no means calculated to open the way to the docks of Amsterdam to the largest-sized ships, although they could pass the new locks at Ymuiden. The stages for loading and unloading vessels will also require important improvements before they are fit for the giants of modern navigation.

It is intended to carry out the necessary improvements and enlargements by easy stages, while it is in no way considered peremptory for all the works to have received, at the time the new lock will be completed, dimensions corresponding to those of the latter. The excessive depth, width and length of that lock have been thus fixed, not with a view to immediate necessity, but are reserved for the "ship of the future." On a preceding page mention has been made of an increase of depth to be given to the channel of the outer canal in the years to come.

In 1889, indeed, the deepening of the canal to 9 m. below A. P. was commenced. The work is now in full progress, and will be quite finished in 1896. This enlargement of the canal is due to the notion that for the nearest future it will be sufficient to expect vessels of the following dimensions: Length, 173 m.; breadth of beam, 15.5 m.; draught, 8 m.

On Plate VI, Figs. 5 and 6, the dimensions of this new canal profile have been given.

As the "ship of the future" is anticipated, in connection with the dimensions of the new lock at Ymuiden, a vessel 220 m. long, 20 m.

broad and having a draught of 9.3 m. On Plate VI, Fig. 7 represents the midship sections of two such vessels abreast in a cross-section of the canal.

If Amsterdam, after a short lapse of time, through ever-increasing prosperity, require the profile here sketched for its canal, millions, indeed, will be needed to bring about that consummation, but they would certainly be well spent where the interests of one of the vital parts of the country are at stake. It is hardly necessary to observe that the before-mentioned bridges across the canal will also have to be widened in the future or will have to be supplanted by other arrangements.

The docks and quays at Amsterdam, partly dating from olden times, but considerably improved after the construction of the North Sea Canal, are superintended by the municipal authorities and by the railway companies concerned. The trade of Amsterdam is charged with the responsibility of keeping its docks and harbor up to the requirements of the present time.

A Commission appointed by the Board of Trade at Amsterdam published a short time ago an important account, proposing numerous improvements.*

Amsterdam is the trysting place of international and inland navigation; an extensive system of canals and railroads connects the city with all the parts of the country. The North Sea Canal itself extends eastward as far as the Schellingwoude Dam, where ships are admitted to the canal connecting Amsterdam with the River Merwede, and where the Orange locks are, day and night, giving a passage to a countless fleet of fishing and other small craft from the Zuyder Zee to Amsterdam and *vice versa.*

The embankment and also the locks at Schellingwoude have already been described in the paper by J. G. Barnard. No change worth mentioning occurred to these works after the completion of the canal.

The principal dimensions of the locks may be here recapitulated.

Great tide lock, containing three pair of flood gates and two pair of ebb gates, width, 18 m.; depth, 4.5 m. below A. P.; length of the lock pond, 96 m.

Northern small lock, containing three pair of flood gates and two

* Report of the Commission for the Encouragement of Transit Traffic *via* Amsterdam to the Board of Trade and Industry at Amsterdam (Amsterdam Municipal Printing Works, 1891).

pair of ebb gates, width, 14 m.; depth, 4.5 m. below A. P.; length of the lock pond, 72.80 m.

The southern small tide lock is exactly the same as the northern.

Outlet sluice containing two pair of flood gates and one pair of ebb gates, width, 10 m.; depth, 4.50 m. below A. P.

Drainage sluices underneath the drainage machine; three openings containing a pair of ebb gates and two pair of flood gates each; each sluice 4 m. wide, 3.80 m. deep below A. P.

An important work is now in course of construction, to protect the water-way outside the embankment at Schellingwoude. The condition of the open Y east of the dam at Schellingwoude has undergone great changes through the construction of this bar. The new enclosed Y and the Wykermeer behind it formerly served as a large basin filling and emptying itself regularly at every tide, and in that way causing see-sawing currents powerful enough to maintain in front of Amsterdam and eastward alongside the northern shore as far as the Durgerdam lighthouse a channel of considerable width and depth. This process having ceased, Amsterdam consequently has been relieved of the great trouble caused by the deposits of silt in its harbors and docks.

But this silt is now being deposited outside the bar, where it finds a favorable place, and it would, if nothing were done, threaten the navigation from and to Amsterdam across the Zuyder Zee with total destruction. A glance at the hydrographic charts and a mutual comparison of these lead to the conviction that the danger is not an imaginary one. The lines indicating depths of 25, 33 and 50 dcm. below low water should here be considered.

Fig. 1, on Plate VII, represents the state of affairs according to the soundings of 1848, before the construction of the embankment at Schellingwoude had been taken in hand. A vessel leaving Amsterdam for the Zuyder Zee, to sail, for instance, to Frisia, could follow these channels offering a draught of water of 75, 50 and 33 dcm. to a point some distance past the Ydaarn. At this point the broad shallow flat began, marked off on either side by a line indicating depths of 33 dcm., called the "Pampus." After having crossed this shoal, the deep was once more gained.

Fig. 2, Plate VII, represents the state of affairs in accordance with the soundings of 1888. Between the Orange locks and the Ydaarn the depth has considerably decreased, and nowhere is a depth of 33

dm. below low water to be found. A few stray patches of the channel between the Orange locks and the Durgerdam light-tower still have a draught of water of more than 25 dcm. below low water.

The same thing may be seen in Fig. 3, Plate VII, being a chart of very recent date, so that part of the bar under construction has already been represented as finished. The construction of this bar may be considered as the opening of a battle against the elements of Nature, fought for the creation and maintenance of a commensurate channel for navigation stretching beyond the Pampus. The dam or bar extends from the Orange locks to a point opposite the Durgerdam light-tower and is composed of two straight parts connected by a curve of a radius of 2 000 m.

The construction of the bar was decided upon after the fact had been incontestably proved that mere dredging in the open Y could not keep up a stationary depth. A single gale appeared capable of destroying the work performed by the dredgers. Hopes are entertained that when the bar shall have been finished a channel may, without great trouble, be cut and maintained in the north side of it, the dimensions of which have provisionally been fixed at: Bottom width, 225 m.; depth below ordinary low water, 2.6 or 2.7 m. The execution of this subdivision of the work will be delayed, however, until there is a call for it, which is as yet not the case.

The formation of the soil in the open Y is very oozy, as may appear from the details on Plate VII, where Figs. 4, 5, and 6 represent the result of three borings performed in the axis of the bar at distances from the embankment of 1 185, 3 304 and 4 250 m. A stratum of very soft clay, here and there intermingled with fen, sand and shells, reaches to 5 or 6 m. below A. P.; a substratum of considerable thickness of grayish-blue clay reaches to 11 or 12 m. below A. P.; then follows sand, with sandwichings of clay, in which fen is also found here and there, on the whole characterized by the absence of shells. At a depth of 17.5 m. below A. P. pure, fine-grained sand makes its appearance.

In laying down the plan for the bar the oozy nature of the soil had to be kept in view, and many arguments led to the construction simply by means of sand. In the first place, the embankments of the outer harbor of the Merwede Canal had been formed in the same way on a foundation soil naturally possessing many features in common

with that of the projected bar. But, in the second place, it seemed a great advantage to draw upon the ground destined for the fisheries harbor at Ymuiden for the necessary sand. In this way one work accomplished two results.

Concerning the quantity of sand required, great uncertainty prevailed. In consulting the experience gained at the Merwede Canal, as before mentioned, 140 or 150 cu. m. per meter length might be assumed as the quantity required. In order, however, to spare the contractors any risk this uncertainty might give rise to, the contract conditions that a quantity of 600 000 cu. m. of sand is to be reserved for the construction, while, in case more or less should be wanted, compensation would be granted on either side at the rate of 40 cents per cubic meter.

The work was contracted for on December 1st, 1889, the execution was commenced in the spring of 1890. In the beginning things got on all right; according to an observation made at a distance of 500 m. from the embankment it appeared that even less sand was required than had been anticipated. Matters suddenly took a different turn, however, when the bar had advanced to a distance of 2 000 m. from the embankment. The soil proved at that point to be of an uncommonly oozy nature, so that four or five times the quantity of sand originally thought necessary was poured in. The occasion seemed to warrant an inquiry into the bearing of the sand upon the foundation soil. The inquiry was performed by means of borings, the result of which have partly been represented on Plate VII in six cross-sections (Figs. 7–12). In the first two sections, at distances of 500 and 1 185 m. from the embankment, the sand has penetrated to 6 or 7 m. below A. P., and on comparison with the results of the borings, especially of boring No. 1, it appears that the so-called "old" grayish-blue clay is of sufficient stability to bear the weight of the sand bar.

The aforesaid unfavorable section was then reached at, to all appearances, a very deep channel silted up in a later period, and a glance at the cross profiles shows that at a distance of 2 320 m. from the embankment the sand has penetrated to a maximum depth of 22 m. below A. P., where it has found a layer of blue coarse sand and gravel to support it. It should further be observed that in order to allow the sand to penetrate as far as possible into the soil, the bar was

primarily raised to a level of 2 m. above A. P. No finish being given to it before the bar had entirely settled, when the surplus of sand was removed.

At a distance of 2 800 m. from the embankment the stability of the soil increases. The depth to which the sand penetrates is there 11.60 m. below A. P. at the utmost, while the quantity of sand required per meter length has come down to 268 cu. m. Recent calculations have led to the supposition that for the whole bar 900 000 cu. m. of sand will be wanted, *i. e.*, one-half more than the original estimate.

Figs. 13 and 14 on Plate VII show the protection of the crown and slopes as originally designed. The crest reaches to 1.20 m. above A. P., and the cross-section presents the form of a shell. The ground plan of the crown and slopes and berms is altogether 13 m. broad, and as it would have been very expensive to cover this whole area along its entire length (of 4 300 m.) and breadth with stone slopings, another means of protection has been resorted to.

The protection now consists of a puddling of clay covered with rushes strewn with brick rubble underlying a stone rubble; 70 tons of stone will be used per 100 sq. m. Concerning the slopes it has been supposed that in some places the pressed-up soil will form a natural berm, making further protection unnecessary, while on those points where the bottom of the sea is of greater stability, fascines would have to be applied.

With regard to this detail the result fell short of the expectations. Berms of pressed-up matter have indeed been everywhere found, but now that some portions of the bar have during two years been settling down, it appears that the berms either miss the desired height here and there or have been gradually washed away by the surf in rough weather. The foot of the stone-faced berms then loses its support, and falls and shifting are the consequences. The protection at the foot had therefore to be extended.

Two systems of protection have been designed. They may be seen on Plate VII, Figs. 15 and 16, and keep the middle between the two extremes conditioned by the contract.

In looking over these profiles it should be borne in mind, however, that the bar has been represented on these drawings without the changes its settling down is expected to bring about. It is hoped that

the bar will be finished in 1893. Inclusive of the compensation for the additional sand, the cost will amount to about 850 000 guilders.

Another important work in course of construction at Schellingwoude is the building of a new drainage machine to be substituted for the present one. This machine will have to contribute to the maintaining of the level adopted for the North Sea Canal. For a part this is done by sluicing through the locks and by sluices at Ymuiden and at Schellingwoude. At Ymuiden this operation is a great nuisance to trade, a nuisance which will not disappear even when the new tide-lock shall be ready.

At the Orange locks the opportunity for letting out water is quite rare owing to the little difference between flood and ebb tide (on an average the former reaches to 0.074 m. above A. P., the latter to 0.31 m. below A. P.). This is especially the case when it is most wanted. Through the influence of weather and wind the Zuyder Zee level may sometimes be kept very high. It has been observed, for instance, that from 1882 till 1887 the average level of the sea at that point reaches 0.41 and even to 0.60 m. above A. P., during periods of four days and nights, under the said circumstances. During the very rare storm tide of December 12th to 16th, 1883, it reached 0.80 m. above A. P.

The quantity of water which may be discharged into the canal varies between 4 000 000 and 7 000 000 cu. m. per 24 hours. According to observations made in 1889, a year which may be called rather unfavorable, the increase of water per 24 hours may amount to 5 908 800 cu. m., and consequently the increase of level of the enclosed Y, covering an area of 1 020 hectares, amount to 0.58 m.

The present drainage machine at Schellingwoude possesses three horizontal turbines, each driven by a separate steam engine, improved in 1886 to 200 I. H. P. During the testing of one of the machines in May, 1887, at a difference of level of 0.43 m., the number of revolutions of the water-shaft per minute was 56.4, and the discharge 491 cu. m. per minute, corresponding to 46.9 H. P., so that the efficiency appeared to be $\frac{46.9}{200}$, or 23 per cent. Consumption of fuel during the testing, 7.5 kg. per hour and per horse-power water lifted, an unfavorable state of things, chiefly owing to the fact that for small lifts turbines are not to be preferred.

At a difference of level of 0.85 m., a common occurrence, when the service of the machine is chiefly required, it can discharge per 24 hours 1 075 000 cu. m. of water, equal to a decrease of level of the North Sea Canal of 10 cm.

The drainage machine now under construction will be 300 H. P., and at a difference of level of 0.85 m., will be able to produce a decrease of level of 22 cm. in 24 hours. The usually small lift here required fixed the choice of apparatus on paddle-wheels. The only fit site for the machine was the space now chosen near the embankment, between the Orange locks and the point of issue of the bar in the open Y now under construction. It is estimated that 577 I. H. P. will be necessary for an efficiency of 300 H. P. water lifted. The number of wheels has been fixed at six, their diameter at 8.5 m., their breadth at 3 m. Immersion 1.55 m. at a level of 0.3 m. below A. P.

On July 30th, 1891, a contract was made for 149 500 guilders, for the building, fitting and maintenance of the engines, the boilers and the paddle-wheels, for a drainage machine at Schellingwoude belonging to the North Sea Canal works. These works are in course of construction. The principal arrangements will be as follows: Two compound engines, having high pressure cylinders 0.62 m. diameter, and low pressure 1 m. diameter by 1.3-m. stroke. Number of indicated horse-power, 289 per engine. There will be six boilers 10 m. long and 2.2 m. in diameter, Cornish system, with internal stoking spaces and Galloway tubes. The proportion of the number of revolutions of the water-shaft to that of the revolutions of the main shaft is 1 to 10, effected by double-tooth wheel-gear. Consumption of steam under normal circumstances, 7.5 kg. per indicated horse-power per hour. Consumption of fuel 1 kg. (of Ruhr coals) per 9.5 or 10 kg. of steam. Efficiency of the machine under average expansion is 300 H. P. water lifted by the wheels from a canal level of 30 cm. below to a sea-level of 0.85 m. above A. P.

While these engines and wheels are constructing at the factory, a sand dumping has been carried out on the site the drainage machine is to occupy in the enclosed part of the Y, in order to improve the foundation soil. This was finished at the end of June, 1892, and cost 72 600 guilders.

On Plate VIII, Figs. *a* to *f*, has been represented the depth to which

the sand has penetrated and pressed away the clay. In order to prevent these borings from causing springs which might afterwards seriously impede the construction of the engine-house, the boring tubes have been left in the holes, and been filled up with Portland cement mortar after the operation.

On July 21st, 1892, the foundations of the engine-house, here referred to, as they were to be laid on the improved soil, were contracted for. The work will cost 89 430 guilders, and is now in course of execution. The chief conditions of the contract are the laying of 2 500 sq. m. of flooring, by means of timbers supported by about 2 200 piles, the top of the floor reaching to 2.72 m. above A. P.

In a short time the building of the engine-house itself will, in all probability, be contracted for.

Plate VIII gives the ground plan as it was designed in connection with the works already in course of execution, together with a front view in perspective, as it was provisionally designed for the building to be erected on this plan.

In order to give an insight into the importance to navigation of the North Sea Canal, a statistical table is given below concerning the traffic through the North Sea locks, and through those at Schellingwoude.

Year.	Number of sea-going vessels having passed the Ymuiden locks in both directions.	
	Number.	Tonnage (cubic meters).
1877.	3 376	2 283 305
1878.	3 242	2 516 310
1879.	4 013	3 008 694
1880.	4 495	3 395 109
1881.	4 603	4 590 321
1882.	4 674	5 175 695
1883.	5 590	5 437 291
1884.	5 435	5 932 362
1885.	5 811	5 962 657
1886.	5 942	6 253 125
1887.	6 256	8 214 732
1888.	6 859	8 653 521
1889.	6 794	8 891 011
1890.	7 752	9 428 112
1891.	8 149	10 227 504
1892.	8 043	10 485 138

Years.	Number of sea-going vessels and river craft having passed the Schellingwoude locks in both directions.		
	Sea-going vessels.		River craft.
	Number.	Tonnage (in cubic meters).	Number.
1884.	120	56 451	84 509
1885.	104	50 668	85 928
1886.	67	38 170	86 172
1887.	56	27 260	89 437
1888.	41	20 950	77 638
1889.	33	13 946	84 709
1890.	20	4 577	79 766
1891.	14	6 577	106 818
1892.	17	3 181	94 063

Canal and Harbor Dues.

From the beginning different dues have been levied on the North Sea Canal, viz., harbor dues, canal dues and stage dues.

These dues have been for the most part abolished by virtue of an agreement drawn up between the State of the Netherlands and the municipal authorities of Amsterdam, Zaandam and Westzaan, ratified by the law of June 28th, 1890.

According to these agreements the Government canal and harbor dues were, dating from July 1st, 1890, abolished for ships bound for or coming from the said places, while the levying of stage dues of those ships was also suspended.

Sea-going vessels or steamers bound for Amsterdam, Zaandam or Westzaan are therefore exempt from all dues.

Appendix.

I.—Dutch measures compared with American and English:

a. Measures of length: Meter (1 000 mm., 100 cm., 10 dcm.) 3.28 ft.

b. Measures of area: Square meter (1 000 000 sq. mm., etc.) 10.76 sq. ft.; hectare, 107 641 sq. ft.

c. Solid measures: Cubic meter (1 000 000 000 cu. mm., etc.) 35.31 cu. ft.

d. Measures of weight: Kilogram (1 000 gm., 100 dkg., 10 hg.) 2.20 lbs. avoirdupois; ton = 0.98 ton English.

II.—Dutch money compared with American and English:

Guilder (100 cents) = about $0.40 = 0.08 £.

Cent = about 0.40 American cent = 0.00083 £.

VOL. XXX, No. 652.
TRANS. AM. SOC. CIV. ENGRS.
KEMPEES ON NORTH SEA CANAL OF HOLLAND
PLATE II.

NOORDZEE KANAAL

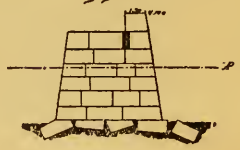
Zee-Haven te Ymuiden.

NORTH SEA CANAL

Samenstelling en Bouw van het Noorder Havenhoofd.

Eerste samenstelling in September 1867.
fig. 1

Eerste wijziging in 1868.
fig. 2.



Wijziging in 1869 tot 1873.
fig. 3.

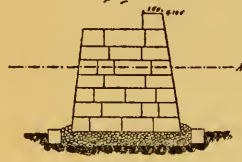
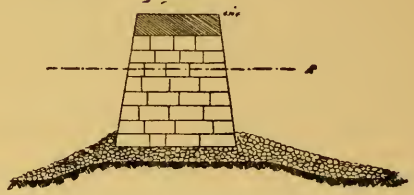
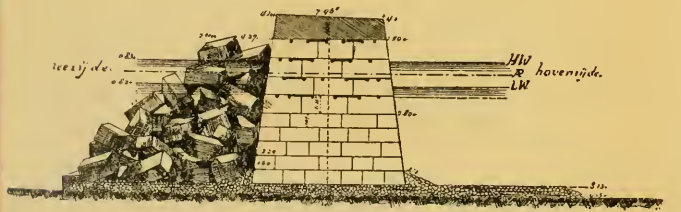


fig. 4

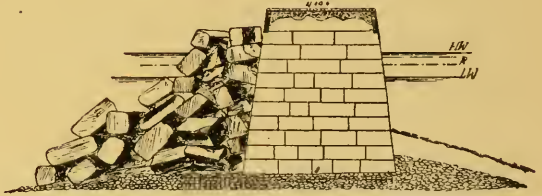


Vernieling door den stormvloed van 2 December 1867.
fig. 5

Samenstelling en Bouw sedert 1873.
fig. 6.

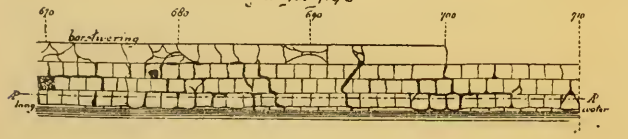


Herstelling van de kruin der Hoofden —
fig 7

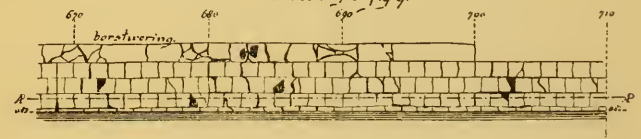


Noorder Havenhoofd van 670 tot 710 M. uit den Duinvoet in Mei 1872.

Zee-zijde fig 8



Haven-zijde fig 9.



TRANS. AM. SOC. CIV. ENGRS
KEMPEES ON
NORTH SEA CANAL OF HOLLAND
PLATE III

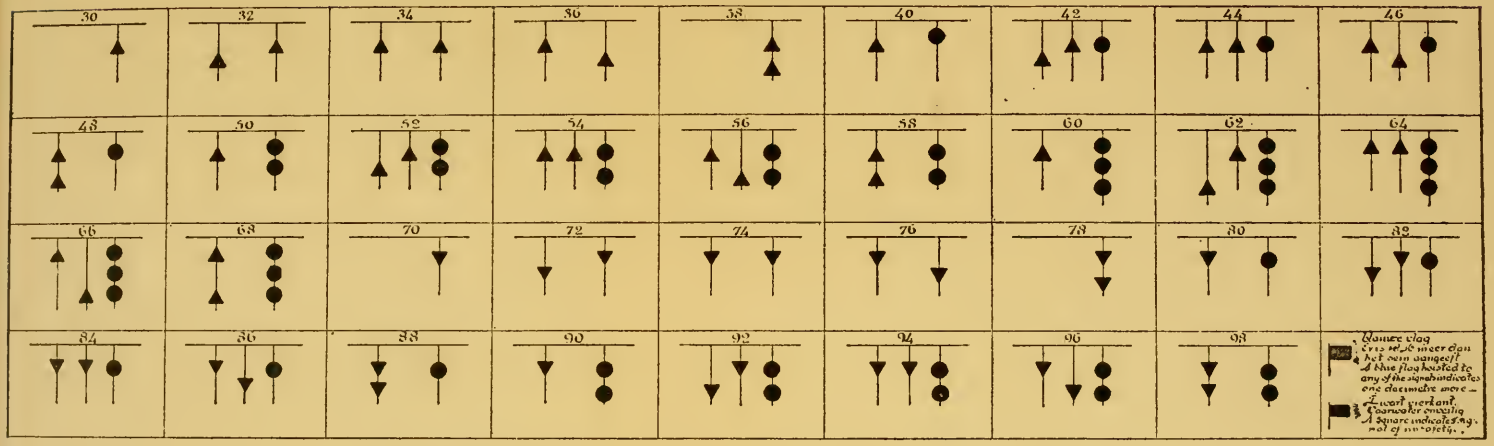
VOL. XXX, No. 652.
TRANS. AM. SOC. CIV. ENGRS.
KEMPEES ON NORTH SEA CANAL OF HOLLAND.
PLATE IV.

NOORDZEE KANAAL

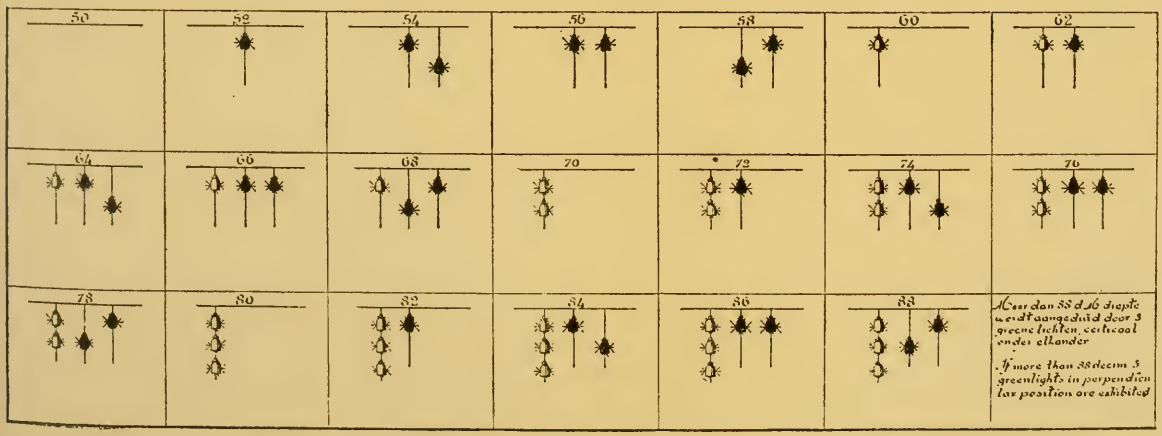
Getij Seinen
Water Signals

NORTH SEA CANAL

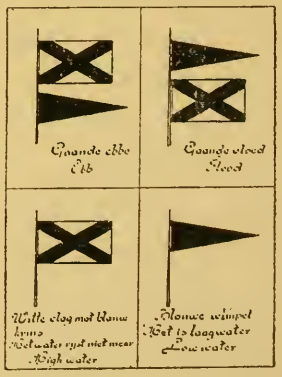
Bij dag met zwarte ballenen kegels (uit zee gezien)
Day Signals (Black balls and cones (as viewed from Sea)



Bij nacht met witte en groene lantaarnlichten (uit zee gezien)
Night Signals White and green lights (as viewed from Sea)



Eb en vloedseinen.
Ebb and Flood signals

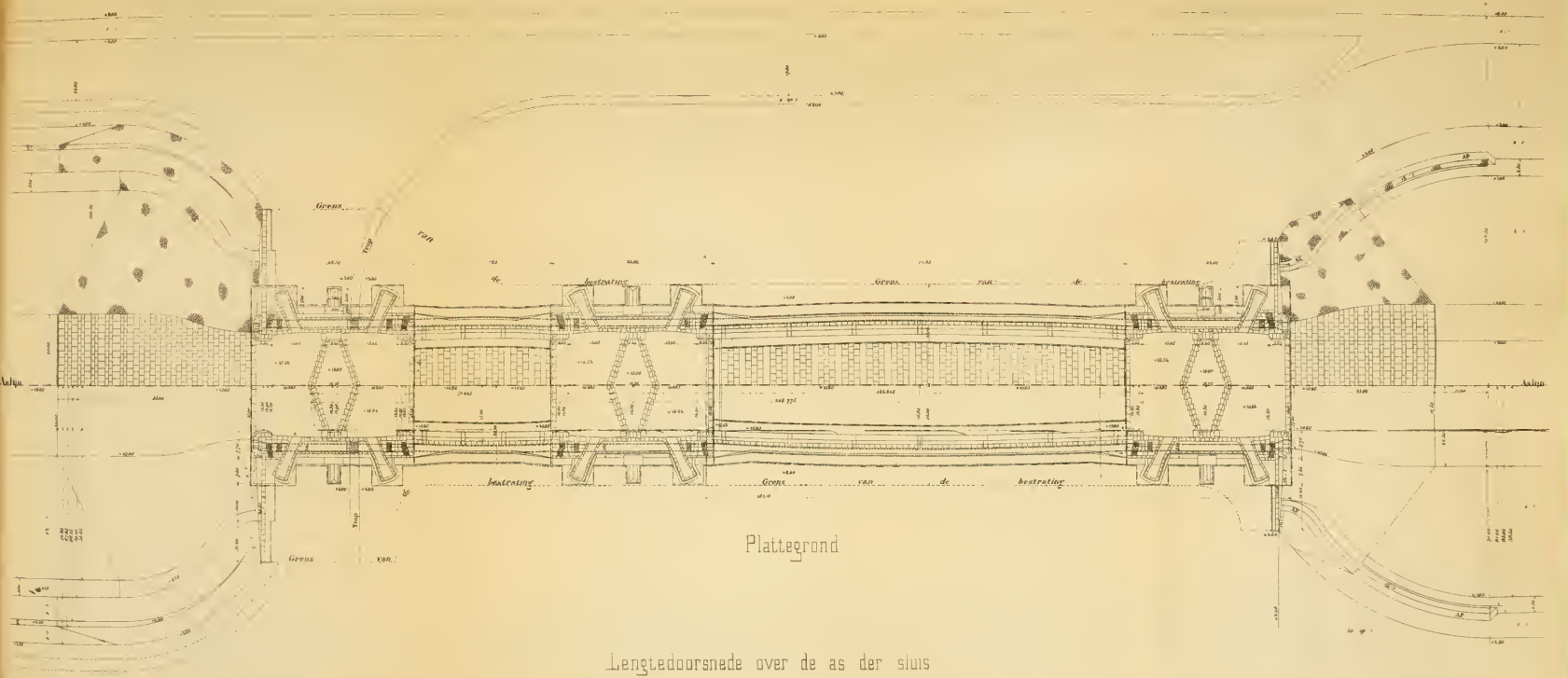


VOL XXX. No 652
TRANS AM SOC CIV ENGRS
KEMPEES ON
NORTH SEA CANAL OF HOLLAND
PLATE V

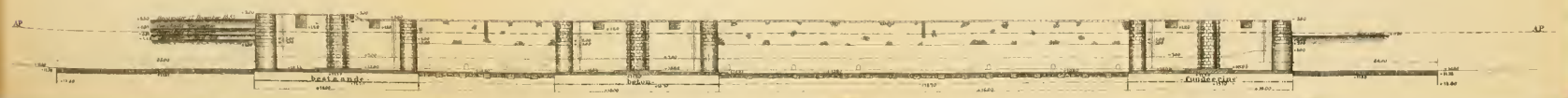
NOORDZEE KANAAL

SCHUTSLUIS TE IJMUIDEN.

NORTH SEA CANAL



Plattegrond



Lengtedoorsnede over de as der sluis

VOL. XXX, No. 652.
TRANS. AM. SOC. CIV. ENGRS.
KEMPEES ON NORTH SEA CANAL OF HOLLAND.
PLATE VI.

NOORDZEE KANAAL

NORTH-SEA CANAL

Dwarsprofillen

FIG. 1. PROFIL VAN AANLEG.

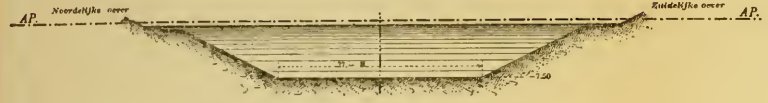


FIG. 2. VAN NOORDZEESLUIZEN TOT ZIJKANAAL A.

FIG. 5. IN BUITENHUIZEN.

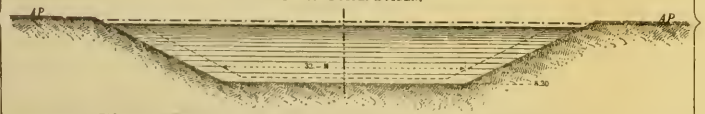


FIG. 4. BEWESTEN BUITENHUIZEN EN VAN BUITENHUIZEN TOT AMSTERDAM

Verordening en verdieping tot 8.30 M. – AP. 1877 – 1883 en 1887.

FIG. 5. VOLGENS HET IN UITVOERING ZIJNDE WERKPLAN (VOLTOOIING IN 1896.)
NORMAAL DWARSPROFIL

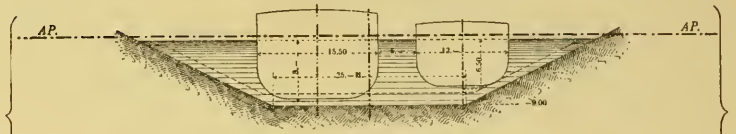


FIG. 6. IN DE WISSELPLAATSEN

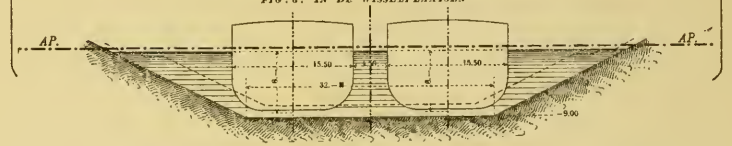
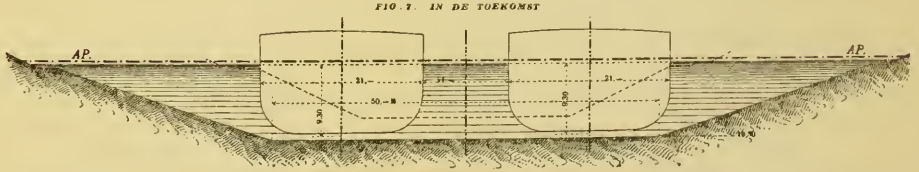


FIG. 7. IN DE TOEKOMST



NOORDZEE KANAAL NORTH SEA CANAL

GRONDBORINGEN TER PLAATSE VAN HET TE BOUWEN STOOMGEMAAL bij

AANDUIDING DER PLAATS

BORINGEN VERRICHT VOOR NA

a b c d e f

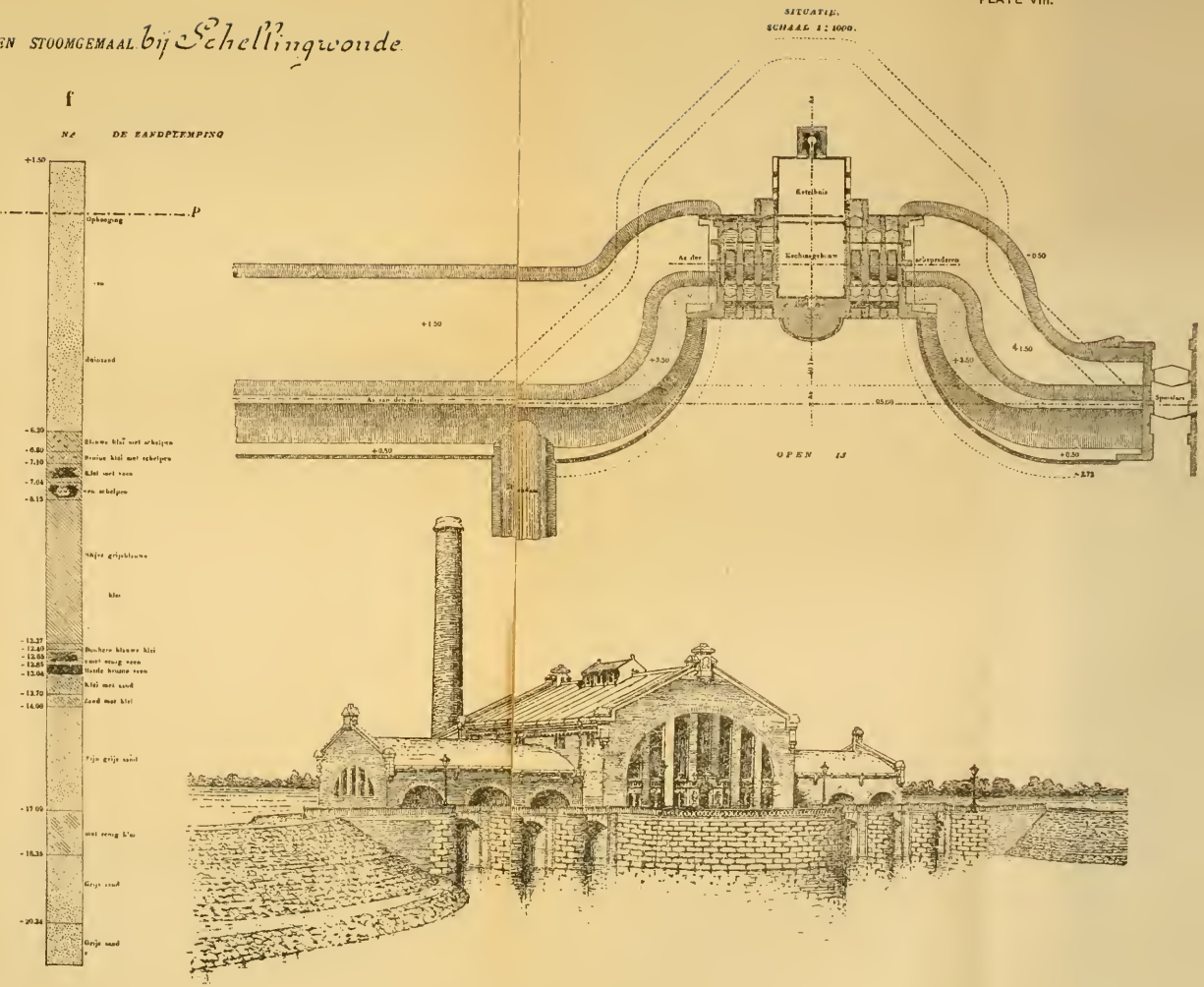
VOOR en NA

DE ZANDPERSMPIN

A P

Water
Ophooging
Zeezand
Dunzand
Sleppe grijze klei
Grijze klei met zand
Harde bruine klei
Harde veen met zand
Harde klei met veen
Tamelijk fijn grijs zand
Bruine klei met zand
Zand met bruine klei
Fijn grijs zand
Klei met weinig zand
Tamelijk fijn grijs zand
Grof scherp zand met grint
Sleppe klei met deeltjes
Sleppe klei met schelpen
Veen met klei
Grijze klei met schelpen
Grijze klei met veen
Grijze klei
Grijze en bruine klei
Harde bruine klei
Veen
Tamelijk fijn grijs zand
Bruine klei met zand
Zand met weinig bruine klei
Fijn grijs zand
Klei met veel zand
Harde grijze klei
Fijn grijs zand
Tamelijk fijn grijs zand
Blauwe klei met zand en schelpen
Grijze klei met bruine veen
Bruine veen met klei
Vette grijze klei
Vette blauwe klei met veen
Blauwe klei met schelpen
Grijze klei met bruine veen
Bruine veen
Sleppe blauwe klei met zand
Sleppe grijze klei met zand
Lichtbruine veen
Grijze klei met zand
Grijs zand met klei
Fijn grijs zand
Tamelijk fijn grijs zand met leem
Fijn grijs zand
Sleppe grijze klei
Sleppe grijze klei met deeltjes
Tamelijk vaste klei met weinig veen
Tamelijk vaste grijze klei
Harde bruine klei
Harde veen met klei en zand
Tamelijk fijn grijs zand
Bruine klei met zand
Fijn grijs zand
Grijze klei met zand
Zand met grijze klei
Tamelijk fijn grijs zand
Blauwe klei
Blauwe klei met zand
Tamelijk fijn grijze zand
Blauw zand met klei
Grijze klei met schelpen
Grijze klei met veen
Grijze klei met weinig veen
Blauwachtig grijze klei
Grijze klei met veen
Bruine veen
Sleppe blauwe klei met zand
Fijn blauw zand
Fijn blauw zand met leem
Fijn blauw zand
Grijs zand vetting
Fijn blauw zand
Ophooging
Zeezand
Dunzand
Sleppe grijsblauwe klei
met veen en schelpen
" met schelpen
Harde bruine veen
" met zand
Tamelijk fijn grijs zand
" met zwarg veen
Grijs zand
met leem
Zand met stijk
Klei met zand en schelpen
Klei met zand
Grijze klei
met veen
Harde bruine veen
" met zand
Fijn grijs zand
Zand met zwarg klei en veen
Zand met donkere klei
Zand met zwarg leem
Grijze zand
Grijze zand
Blauwe klei met schelpen
Bruine klei met schelpen
Klei met veen
veen schelpen
Grijze grijsblauwe klei
Donkere blauwe klei
met zwarg veen
Harde bruine veen
Klei met zand
Zand met klei
Fijn grijs zand
met zwarg klei
Grijs zand
Grijze zand

De dieptematen zijn uitgedrukt in Metres ten opzichte van Amsterdamsch Peil


EN STOOMGEMAAL bij Schellingwoude
f
NA DE ZANDPLEMPING
SITUATIE.
SCHAAL 1 : 1000.
Ketelhuis
Kuchengebouw
As der
schepsteenen
As van den dijk
Sporstuur
OPEN 15
+1.50
+3.50
+0.50
-0.50
-2.73
Ophooping
Blauwe klei met schelpen
Bruine klei met schelpen
Klei met veen
Zand met klei
Fijn grijs zand
Grijs zand
-6.30
-6.80
-7.10
-7.64
-8.15
-13.37
-12.40
-12.65
-12.85
-13.04
-13.70
-14.00
-17.09
-18.35
-20.24

List of Plates.

Plate I. Strategical map.
" II. Construction of the piers at Ymuiden.
" III. Hydrographic chart giving a view of Ymuiden Harbor.
" IV. Day and night signals at Ymuiden.
" V. New locks at Ymuiden (in course of construction).
" VI. Cross profiles of the North Sea Canal.
" VII. Embankment bar in the open Y (in course of construction).
" VIII. Steam drainage machine at Schellingwoude (in course of construction).

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

653.

(Vol. XXX.—November, 1893.)

NOTES ON PROJECTS FOR THE NORTH SEA CANAL FROM 1629–1893.

By A. HUET, C. E.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

The United Netherlands owed their political independence in the 17th century in a large measure to the prosperity of the Dutch sea trade, which in those times embraced nearly one-half of the European commerce. The chief center of its extensive shipping was the rich city of Amsterdam, capital of the prosperous province of Holland and situated on the south side of a creek or inlet of the Zuider Zee, called the Y. This creek offered, over a short length, a depth of 30 ft., but opposite the city the bottom silted up and the entrance to the creek was barred by a shoal, called Pampus, which hardly offered 10 ft. of water.

To get rid of the mud banks in the port, Burgomaster Hudde in the 17th century invented the mud-dredger, prototype of the

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

present dredging machines; and the passage of the bar was made possible by the invention of "the ship camel," a kind of floating caisson which was attached to the side of a ship and lifted her over the shoal.

In 1825 a canal of 80 km. length, parallel to the sea coast, was made to connect Amsterdam to the port of Nieuwe-Diep, and some time afterwards sluice docks were constructed to provide for better ship accommodation at Amsterdam. These works proved sufficient in their days, but the advent of steam navigation necessitated the opening of a shorter route to the North Sea.

A series of projects for a canal direct to the coast were made, which resulted in the construction of the present North Sea Canal, with locks on both ends, opened for commerce in 1876.*

First.—The projects from 1629–1823, which were intended to connect the Y to the North Sea for other objects than the commercial wants of Amsterdam.

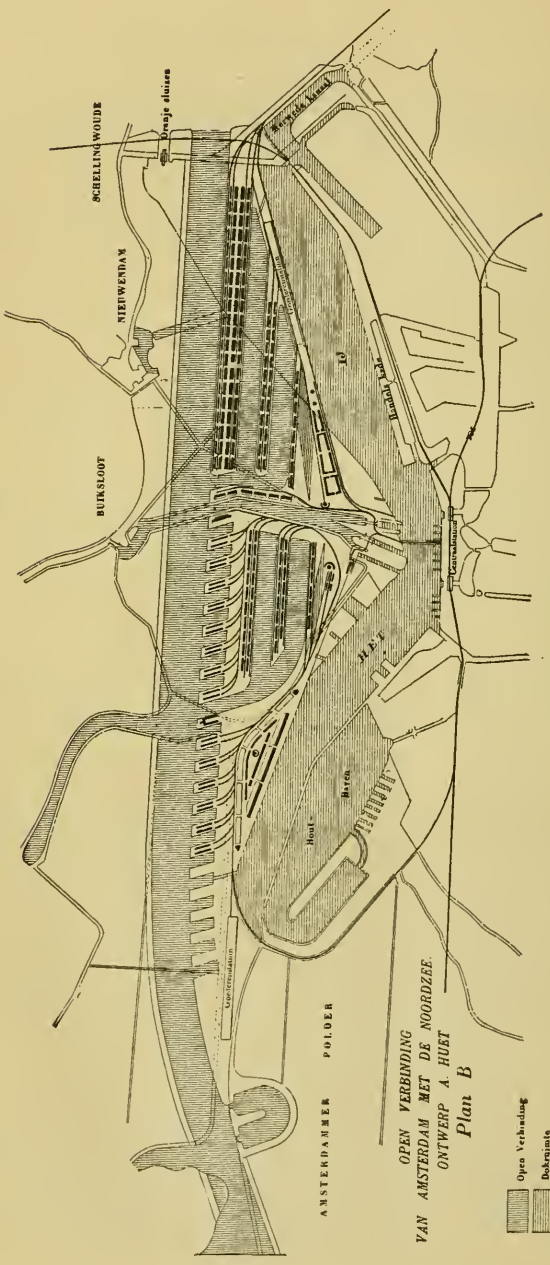
Second.—A large number of projects from 1849–63 for canals with locks at both ends, between Amsterdam and the North Sea.

Third.—A few projects from 1863–74 for an open cutting between the North Sea and the Zuider Zee to provide a free access to Amsterdam.

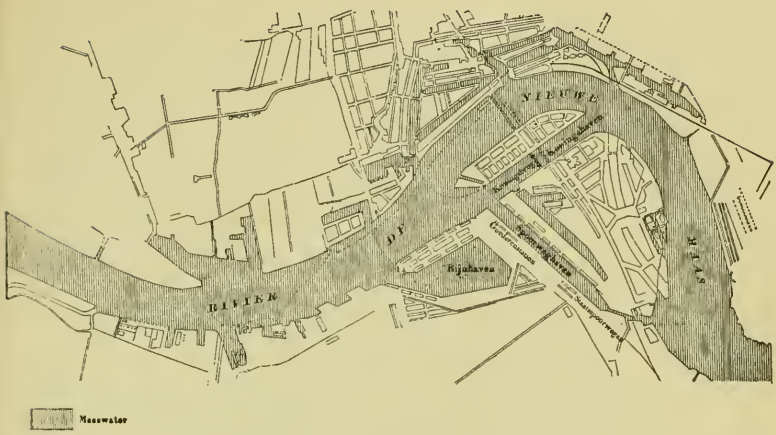
The author of this notice visited Liverpool in 1861 and was profoundly impressed by the enormous extension of trade and shipping he witnessed in that port. He was brought to consider that a perfectly free access to the Atlantic Ocean was a *conditio sine qua non* for the progress of commerce in Amsterdam. In 1862 he obtained a vote of the Congress of the "Netherland Society of Industry," that the possibility of an open sea canal for Amsterdam ought to be seriously examined. A sketch design was given by him in 1863. In the meantime a concession had been given for a canal with sluices, but the works progressing slowly in 1867 and the opening of the Suez Canal which had no sluices, being at hand, the author obtained from the above-named society the appointment of a commission to examine if an open sea canal was possible and desirable.

The canal with locks was finished, as stated above, in 1876, but in the meantime the dimensions of ocean steamers had so much augmented that very soon (in 1886) larger locks proved a necessity, and such are at

* A complete survey of these projects as far as they were published is given in a paper read before the Institution of Civil Engineers in 1889, and offered by the author to the Congress (No. 2).



AMSTERDAM (ONTWERP).



ROTTERDAM.

HAMBURG.

present in course of construction. The author urged the desirability of an open cutting from Amsterdam to the sea several times in the Institution of Civil Engineers and read a paper in 1884 on this subject.* The debates on this subject ended in 1890 by the Institution inviting the author to make a preliminary project of an open sea canal and by giving him the assistance of an Advisory Commission chosen from the members of the Institution. The president and secretary of this commission were Messrs. T. P. R. Tale van Poortolich and C. Lely, who sometime afterwards became Ministers of Interior Affairs and of Public Works respectively.

This preliminary project was established in an assembly of the Institution on February 14th of this year and is now in course of publication. The principal feature of this preliminary project is an open canal between Amsterdam and the North Sea; length, 28 km.; breadth, 300 m., and depth, 10.82 m. on high water-mark.†

In close relation to this open canal are the projected harbor works for Amsterdam, of which illustrations are given herewith. The illustrations give a comparative view of the harbors of Hamburg and Rotterdam and the projected harbor works of Amsterdam. The intention of the project is to have harbor accommodation for the largest steamers afloat, as the tides between the North Sea and the Zuider Zee insure sufficient and constant depth of water.

There can be hardly a doubt that these works, if executed, would provide a safe and profitable terminal for a direct line of first-class steamers between the United States and the Continent of Europe, which would complete advantageously, the new American line from New York to Southampton.

Close relations have always existed between the United States and the United Netherlands who founded New Amsterdam, and the sympathy between the two nations was sufficiently proved by the loan of $35 000 000 subscribed by the Dutch during the War for Independence.

It would certainly prove a great boon for both nations if it might be possible to enable old Amsterdam to harbor a first-class line of steamers running between New York and Amsterdam, bearing the stars and stripes.

---

* A copy of this is offered to the Congress (No. 1).

† A copy of this will be offered to the Congress (No. 3) as soon as it is published.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

654.

(Vol. XXX.—November, 1893.)

THE NORTH AND EAST SEA CANAL.

By J. FÜLSCHER, Geheimer Baurath, Member of Imperial Board of Canal Commissioners at Kiel, Germany.

Translated from the German by KENNETH ALLEN, M. Am. Soc. C. E.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

PART I.

HISTORY OF ITS INCEPTION, CONSTRUCTION, AND ITS IMPORTANCE IN UTILITARIAN AND MILITARY ASPECTS.

1. *History of the Canal.*—The project of constructing a water-way between the North and the East seas, which would avoid the tedious and dangerous voyage around Skagen, and connect the two seas more closely in a manner particularly advantageous to Germany, has been discussed for more than five centuries. Three of the projects were actually carried out, but a large number never passed beyond the preliminary work, or the first stages of construction. The completed works are, 1st, the Strecknitz Canal, between the Trave and the Elbe, built in the years 1391–98; 2d, the Alster-Trave Canal, built in 1525;

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

3d, the Eider Canal between Holtenau (Kiel) and Tönning, built in the years 1777–84.

The first two canals, of which the Alster-Trave Canal was destroyed in the year 1550, should truly be classed with those inland canals which are not adapted to maritime traffic, even under the conditions formerly attending navigation.

The natural course of the Eider from Tönning to Steinrade was made use of in the Eider Canal, to connect the two seas ; and from thence a new water-way was made which included the upper Eider as far as Flemhuder Lake, and then, after breaking through the water-shed north of the city of Kiel, reached the creeks of the same name.

On account of its slight depth, its many windings and its six locks, this canal could aid navigation but slightly. It was therefore the earnest wish of merchants and navigators to inaugurate a change.

These projects served military and political ends as well as assisting commerce, especially where they were promoted by the Germans. On this account none of them have succeeded under the Danish kings, who held entire control of the lines of commerce through the Sund and the Belts.

In the year 1864, after the separation of Schleswig-Holstein from Denmark, the Prussian Ober-Baurath Lentze was commissioned by the Prussian Minister of Commerce to make the preliminary examinations for a North and East Sea Canal.

In the project prepared by Lentze, St. Margarethen-Rendsburg-Eckernförde, which, at the request of the Prussian Minister of War, was changed to St. Margarethen-Rendsburg-Kiel, the canal for the first time assumed the shape of a cut at sea-level with but one lock, at the point where it enters the Elbe. On account of the political events of the year 1866, this plan was not carried out.

Finally, in the year 1878, the final period in the preliminary history of the canal began, when a Hamburg merchant and ship-owner, H. Dahlström, published a document (The Conditions of a Schleswig-Holstein Maritime Canal requisite for Revenue. Hamburg, 1878), and later received permission from the Prussian government to make the preliminary investigations for a line between Brunsbüttel-Rendsburg-Kiel.

In May, 1881, Dahlström presented the preliminary plans for a canal from Brunsbüttel, through Wittenberge, Rendsburg, Steinrade and

Knoop to Holtenau at the bay of Kiel, which he had prepared with reference to their technical and economical aspects. The plans prepared by Lentze, which he voluntarily placed at Dahlström's disposal, served as a most essential foundation. In them the Inspector of Waterways, Boden, who had been engaged to do the technical work, made some important alterations, which were found necessary by later experience and research.

It was this revised plan of Lentze which (with many alterations, especially influencing the cost) was used as foundation by the Legislative bodies—the Reichstag and the Prussian Landtag.

The construction of a maritime canal, suitable for the German flotilla from the mouth at the Elbe *via* Rendsburg to the Bay of Kiel, at the cost of the German Empire, was approved by law, May 16th, 1886, under the condition that Prussia should contribute 50 000 000 marks in advance towards the 156 000 000 marks, the total cost. This condition was agreed to by Prussia, July 16th, 1886.

On July 17th, 1886, a special commission was appointed, under the jurisdiction of the Department of the Interior, and entitled The Imperial Canal Commission, to carry out the building of the canal, with headquarters at Kiel.

The building of the canal itself was inaugurated by the laying of the corner-stone on June 3d, 1887, by the Emperor of Germany, William I. A period of eight years was allowed to complete it, so that, according to the programme, the North and East Sea Canal should be opened in the year 1895.

2. *The Line of the Canal* (see the accompanying map).—The North and East Sea Canal runs from the mouth of the Elbe, *via* Rendsburg, to the bay of Kiel, and its entire length is 98.65 km. (61.31 English miles). Starting on the west at Brunsbüttel, the canal at a distance of 20 km. (12.43 English miles) passes through the territory of the Elbe Marsh, which at many places is below the water-level of the proposed canal, and through the lowlands of the Burg-Kuden Lake, arrives then by a slight rise to the water-shed between the Elbe and the Eider at Grünenthal, at an elevation in round numbers of 24 m. (78.72 English ft.), and reaches the lowlands of the Eider at 40 km. by following the valley of the Gieselau.

Traversing the valley of this river where subject to inundation, the included moors, and the Meckel Lake, the canal approaches the Eider

for a short distance at Schülp, at 55 km., but does not reach the actual river-bed. A dike is built above high water-mark, between the Eider and the canal, to protect the latter from floods.

The valley of the lower Eider is left again at Westerrönfeld, 59 km.

The canal passes around the city of Rendsburg on the south side, reaches the upper Eider Lake at 65 km., and follows it up to 71 km. From here it passes into the territory of the old Eider Canal, whose course was used as much as possible, although with important straightenings and shortenings. It ends at Holtenau, in the harbor of Kiel.

In the entire length of the canal of 98.65 km. (61.31 English miles) there occurs a fall on—

Straight reaches								62.15 km.	(38.62 miles)	= 63.0%
Curved	"	6 000 m.	(19 680 ft.),	radius	4.82	"	(3.00	")	=	4.9%
"	"	5 000 "	(16 400 ")	"	5.52	"	(3.43	")	=	5.6%
"	"	3 000 "	(9 840 ")	"	13.23	"	(8.22	")	=	13.4%
"	"	2 500 "	(8 200 ")	"	3.77	"	(2.34	")	=	3.8%
"	"	2 000 "	(6 560 ")	"	0.98	"	(0.61	")	=	1.0%
"	"	1 700 "	(5 576 ")	"	1.99	"	(1.24	")	=	2.0%
"	"	1 500 "	(4 920 ")	"	2.99	"	(1.86	")	=	3.0%
"	"	1 000 "	(3 280 ")	"	3.20	"	(1.99	")	=	3.3%
								98.65 km.	(61.31 miles)	= 100.0%

3. *The Profile of the Canal* (see the accompanying map).—The canal is shown here in a section. In order to cut it off from the variable water-levels of the Elbe and the East Sea, locks are built at both ends. The mean water-level in the canal is about the same as the mean water-level in the harbor of Kiel, = —0.23 (0.75 ft.).

In the Elbe, at the western terminus of the canal, where the stage of the water is affected by the regular ebb and flood, the mean high-water level lies at + 1.29 (4.23 ft.), the mean low-water level at — 1.50 (4.92 ft.).

The lock in the Elbe at Brunsbüttel will generally stand open during the ebb from the time when the two water-levels are equalized until the beginning of the flood. Vessels can go freely in and out with every tide for from three to four hours.

The lock at the East Sea will, on the other hand, generally stand open, in consequence of the fact that the variations in the water-level in Kiel harbor are usually slight, and will only be shut when the water-level outside rises higher than 0.5 m. (1.64 ft.) above mean water, or falls an equal amount below mean water. Both conditions may be looked for together in about 25 days of the year.

The canal has a depth of 9 m. (29.52 ft.) below mean water. The bed is horizontal from the eastern terminus at Holtenau to Rendsburg, 60 km. From here to the western terminus at Brunsbüttel the bed has an increasing fall of from 1 : 200 000 to 1 : 25 000, according to the fall of the surface which is formed at the ebb when the gates of the lock at Brunsbüttel are open.

Dikes are built up on both banks where the canal traverses lowlands which lie partly below the surface of the canal.

4. *The Cross-Section* (see the accompanying map).—The bed of the canal in the straight reaches and the curves of over 2 500 m. (8 200 ft.) radius has a width at the bottom of 22 m. (72.16 ft.), and up to 3 m. (9.84 ft.) above the bed a 3 to 1, and, to 4 m. (13.12 ft.) height, a 2 to 1 slope. The upper portions of the profile are built differently for the reaches in the lowlands from those in the higher lands. In the latter reaches, a berm at a height of 7 m. (22.96 ft.) above the bed, that is, about 2 m. (6.56 ft.) below the mean water-level, connects with a 1½ to 1 slope protected by stone, which reaches to 1 m. (3.28 ft.) above the usual water-level. At this height lies a second berm 2.5 m. (8.2 ft.) wide which joins the excavated slope of 1½ to 1.

In the lowlands the lower berm is widened on both sides, in order that the cross-section may be widened later without disturbing the dikes.

The dikes, which are to be built on either side, are given slopes of 2 to 1 where the underlying ground has sufficient firmness—natural or artificially increased by laying sand; but where long stretches have a soft foundation, in order to distribute the pressure as much as possible, they are given an inner slope of 6 to 1.

The cross-section of the canal has, at the lowest stage of water, 6.17 m. (20.24 ft.) depth below the surface, an available width of 36 m. (118.08 ft.), and offers a passage to the largest steamers employed in the East Sea traffic, which, with a few exceptions, have a draft of not over 6 m. (19.68 ft.), with a berm of 12 m. (39.36 ft.).

In curves of from 1 000 to 2 500 m. radius (R) a widening of the cross-section is given of $26 - \frac{R}{100}$ m. This widening amounts to, in the curves, of—

1 000 m.	(3 280 ft.)	radius.....	16 m. (52.48 ft.)
1 500 "	(4 920 ")	"	11 " (36.08 ")
2 000 "	(6 560 ")	"	6 " (19.68 ")
2 500 "	(8 200 ")	"	1 " (3.28 ")

5. *Facilities for Passing.*—Opportunity for the passage of large ships of war is provided by six places arranged for passing, besides the upper Eider Lake, an average distance apart of 12 km. (7.46 miles). These passing places are given, including the passages, a length of 450 m. (1 476 ft.), and a width at the bottom of 60 m. (196.8 ft.). In the Audorf Lake, east of Rendsburg, there is, moreover, a place where vessels of any size can turn around.

6. *Locks.*—Locks at both terminals are arranged as double locks, each with two openings—one for outgoing and one for incoming vessels. Each opening is given a clear width of 25 m. (82 ft.), and a chamber length of 150 m. (492 ft.).

In order to connect the canal with the lower Eider, a lock will be built near the city of Rendsburg with a width of 12 m. (39.36 ft.), and a length of chamber of 68 m. (223.04 ft.).

7. *Bridges and Ferries.*—Of the four railroads which cross the canal, two, the Itzehoe-Heide and the Neumünster-Rendsburg, cross on drawbridges of unequal spans, with 50 m. (164 ft.) clear span; and two, the West Holstein and the Kiel-Flensburg, cross on high bridges.

Of the latter, the bridge at Grünenthal, which is already finished, has a length between supports of 156.5 m. (513.32 ft.). It is constructed with chords arched in the form of a sickle, on abutment joints, and the track is half the height of the arch.

The high bridges have each a clearance of 42 m. (137.76 ft.) above mean water-level.

The drawbridges, which are moved by hydraulic power, are uniformly constructed with a single track, so that, in the case of the double-track road from Neumünster to Rendsburg, each track is carried across on a special drawbridge.

The highway between Rendsburg and Itzehoe, which is much traveled, crosses the canal also with a drawbridge, constructed similarly to the railway spans.

Beside these, ferries are provided for wagon roads. There are fourteen ferries, making one for each 7 km. (4.35 miles).

8. *Housing and Care of the Workmen.*—The housing and care of the workmen is undertaken by the Government.

The buildings for this purpose consist of camps of barracks for from 100 to 350 men, and of single barracks for 50 men each. Each camp consists of a government building, one or more dormitories and

the necessary out-buildings. All the buildings are framed and of one story. The Government building contains, besides a government office and a store for selling food and clothing, a dining and assembly hall of 180 sq. m. (1 936.8 sq. ft.), a basement with kitchen, pantry, laundry, disinfecting-room, bathroom, physician's room and waiting-room.

The dormitories have rooms which can be heated, containing eight cots each, with 12 cu. m. (423.84 cu. ft.) of air space per capita, and one or more rooms for inspectors.

Workmen who become sick or are injured are treated in the public hospitals at Brunsbüttel and Rendsburg, in the Academical hospitals at Kiel, and in two hospitals at Burg and Hanerau, specially erected by the Canal Commission, and governed by their own officials. Especial arrangements are made for taking the men into the first-named hospitals and into the Academical hospitals.

9. *The Execution of the Work.*—The preliminary work for the canal began in October, 1886. It had progressed so far in the course of a year that they were enabled to acquire the right of way in the fall of 1887, and to make contracts for the earthwork in the first half of the year 1888.

Excavation was begun for a short distance in the vicinity of the western terminus in March, 1888, and was continued in October of the same year. The construction of the locks was begun in the fall of 1890, and of the bridges in the spring of 1891.

The time appointed for the completion of the canal was the year 1895.

The total quantity of earth to be moved consisted in round numbers of 80 000 000 cu. m. (2 825 600 000 cu. ft.). Up to the present time, April 1st, 1893, about 60 000 000 cu. m. (2 119 800 000 cu. ft.) have been moved. The monthly average in the year 1892 was, approximately, 1 400 000 cu. m. (49 450 000 cu. ft.).

For the removal of earth, there were in use in the year 1892:

20 land excavators.
42 floating dredges and excavators.
65 locomotives.
1 790 wagons.
662 carts and wheel-barrows.
11 steam scows.
124 transport scows (sloop rigged).
37 tugboats.

The number of workmen employed averaged, in the winter and fall months of 1892, 5 785; in the spring and summer, 8 270.

10. *The Cost of the Canal.*—The entire cost of the canal was estimated at 156 000 000 marks, or, for each kilometer, 1 580 000 marks (1 mile = 2 544 450 marks = £124 728). Judging by previous and existing conditions, we can safely assume that this will not be exceeded.

11. *The Significance of the Canal.*—The significance of the canal is both military and economic. The military significance consists in having a connection between the North and East seas for the German fleet, independent of foreign countries, so that they may pass unhindered and unseen from the East Sea to the North Sea and *vice versa.*

The economic significance of the canal consists, firstly, in shortening the distance between the two seas, as compared with the present route; and, secondly, in avoiding the dangerous voyage around Skagen, thereby preventing the numerous losses of valuable merchandise, vessels and lives which occur every year in these waters.

The shortening of the route depends, naturally, upon the situation of the various harbors.

As a point of departure for an estimate of the shortening of distance between the harbors of the North and East seas, we take a point between the Peninsula of Wittow, which belongs to Rügen, and Torp, on the southern coast of Sweden, in latitude 55° north, longitude 13° east, from Greenwich, from which the routes of navigation from the East Sea follow a common line through the Sund, around Skagen and through the Skagerrak to the North Sea, and from the Skagerrak radiate towards the different harbors. The shortening of the route between the harbor cities for traffic between the North Sea harbors and the East Sea harbors lying east of a line from Wittow to Torp can be seen by the accompanying summary showing the shortening of time and distance. For Leith, on the west coast of Scotland, this is 83.8 nautical miles, and for Hamburg, 424.8 nautical miles.

If we consider the rate of speed of steamers in the open sea to be $8\frac{1}{4}$ nautical miles, and in the canal to be 10 km. or $5\frac{3}{4}$ nautical miles, per hour; and, moreover, if we allow for the unavoidable stops in going through the locks and bridges, in passing each other, etc., three hours for each trip through the canal, we have the gain of time by use of the canal indicated in the above-mentioned summary.

This gain is much greater for sailing vessels, and, with adverse

winds, may amount to weeks. On an average, it would be about three days. In addition to this gain in time, which corresponds to a gain in freightage, as well as in interest due to the quicker delivery of goods, there is a further economy in using the canal in place of going around Skagen; first, in avoiding the cost of pilotage in the Sund; secondly, in a lessening of insurance for vessels and goods when the voyage is made by way of the more safe canal; and, thirdly, in a saving of the money that would be required to run into a harbor in the Skagerrak and Kattegat, especially in winter.

As an offset to this saving, we have the tariff for the use of the canal, which is so adjusted that there results a gain for each vessel using the canal. In addition, there is the advantage accruing to navigation by the lessened danger consequent on the use of the canal as compared with the voyage around Skagen.

According to authoritative inquiries made up to the year 1866, 200 vessels were yearly wrecked in sailing around Skagen, whose value, aside from the great loss of life, is estimated, after deducting the value of the goods taken on commission, at 14 000 000 marks. Allowing for a decrease in loss on account of the increased use of steamers, there yet remains an annual loss estimated at 8 000 000 marks. The greater part of this loss is avoided by using the North and East Sea Canal, and it is an advantage to those nations which participate in the traffic of the East Sea.

12. *The Prospective Traffic of the Canal and its Capacity.*—The traffic through the Sund, the two Belts and the Eider Canal, serves as a basis upon which to estimate the probable traffic on the North and East Sea Canal.

According to the statistics of the five years 1877–81, the marine traffic of the German, Russian, Finnish and Swedish harbors in the East Sea and harbor of Copenhagen, together with harbors outside the East Sea which would profit by use of the canal, averaged 24 000 steamers and sailing vessels, with an approximate tonnage of 8 300 000. As, however, many vessels which might use the canal to advantage will undoubtedly choose the voyage around Skagen, if they are in no hurry, or meet with favorable winds and weather, the annual traffic on the canal is estimated, in formulating the memorial for the canal presented to the German Reichstag, at 18 000 vessels, with a tonnage of 5 500 000.

Since this time, however, the traffic has materially increased, and there is, moreover, an important change in the relative proportion of sailing vessels and steamers. While, according to Dahlström, in "The Arrangements for the Building and Maintenance of the North and East Sea Canal," the traffic through the Sund in the year 1880 consisted of 28 211 sailing vessels and 9 876 steamers, with a combined tonnage of 12 718 376, in the year 1889 there were much fewer sailing vessels and more steamers. The traffic in this year consisted of 14 114 sailing vessels and 16 448 steamers, with a combined tonnage of 16 022 069.

In these nine years there has been, therefore, an increase in the volume of marine traffic in the Sund of—

$$\frac{16\,022\,069 - 12\,718\,376}{12\,718\,376} \times 100 = 26\%.$$

The number of steamers has increased by—

$$\frac{16\,448 - 9\,876}{9\,876} \times 100 = 67\%.$$

The number of sailing vessels has decreased—

$$\frac{28\,211 - 14\,114}{28\,211} \times 100 = \text{about } 50\%.$$

This averages yearly, therefore, an increase in volume of $\frac{26}{9} =$ about 3%; an increase of steamers of $\frac{67}{9} = 7.4\%$, and a decrease of sailing vessels of $\frac{50}{9} = 4.5$ per cent.

Assuming that this change in the traffic in the Sund is not discontinued in the year 1889, but keeps on in the same ratio, then in the year 1895, the first year of operation of the North and East Sea Canal, the traffic on the Sund will have attained the following proportions:

The total tonnage of vessels will be—

$$12\,718\,376\,(1 + \overline{15 \times 0.03}) = 18\,441\,642 \text{ registered tons,}$$

the number of steamers—

$$9\,876\,(1 + \overline{15 \times 0.074}) = \quad 20\,838 \quad \text{“} \quad \text{“}$$

and the number of sailing vessels—

$$28\,211\,(1 - \overline{15 \times 0.045}) = \quad 9\,169 \quad \text{“} \quad \text{“}$$

In a pamphlet recently published in Kiel by the Geheimer Kommerzienrath Sartori, entitled "Kiel and the North and East Sea

Canal," it is shown that, of the traffic on the Sund in 1887, probably 60% would have passed through the North and East Sea Canal if it had been in existence at that time. Taking this percentage as good for the year 1865, the traffic on the canal during its first year of operation (estimating a full year) will be—

	0.6 ×	18 441 642	= approximately	11 600 000 tons
In steamers.....	0.6 ×	20 838	= "	12 500 "
And sailing vessels..	0.6 ×	9 169	= "	5 500 "

This traffic on the North and East Sea Canal can be easily accommodated with continuous operation, day and night, as the provisions for navigation reach far beyond present necessities, and we cannot err in the assumption that, even with a uniform increase of navigation, the canal will suffice, in its projected capacity, to accommodate the traffic for any conceivable period.

PART II.

Construction of the Canal in the Lowlands of the Burg-Kuden Lake.

General Conditions, Preparatory Work and Designs (with a map, Plate I, Fig. 1, and profile, Plate II, Fig. 2).—From 7.6 to 18 km. the canal crosses an extensive low marsh which, in the last century, served as a collecting ground for the high waters of Holstenau, which were injurious to the Wilstermarsch, although they then had an outlet toward the Elbe; but not until the end of the sixties and beginning of the seventies of this century were they improved by carrying out a systematic scheme of drainage. The profile of the canal here shows a section almost everywhere in a soft and submerged soil for its entire depth. On the top, after the removal of turf to a few decimeters in depth, lies boggy soil (moss peat) in variable thickness; below that, darg (sedge peat), and, lowest of all, clay (clammy marsh soil). The old solid sea-bed, from 9 to 16 km., lies everywhere below the canal-bed, and but little higher at both ends of the lowlands.

In order to arrive at a safe opinion of the supporting power of the top stratum of the soil, experimental excavations were made in several places during the preliminary work. It became evident that, even in the two top strata, slopes of 4 to 1 for a slight depth began to settle without a surcharge, so that, even under a light weight, the clay

became compressed and squeezed out. A surcharge on the banks of the canal was unavoidable, because the highest stage of water in the canal + 20.27 m. (66.49 English feet)* lies some 1.25 m. (4.10 ft.) above the mean elevation of the lowlands, and, therefore, dikes had to be constructed on both sides of the canal, to protect the lowlands.

The most unfavorable of these conditions occurred from 13.2 to 18 km., where the stratum of bog was so thick and so soft that it could scarcely be trodden upon, and where the clay also was less firm than elsewhere. Here it was to be predicted with certainty that, with the excavation for the canal, and especially the construction of the above-mentioned dikes, the material at the sides would always slide. Nothing remained, therefore, but to consolidate the soft material on both sides of the canal, or to press it back by filling in with some firmer material, forming in this way solid bodies acting as dams between which the canal could be excavated without further difficulty. A sandy soil adapted to the construction of these dams was found in abundance in the tract beyond the lowlands, from 18 to 26.2 km. In the excavation of the canal between these sand dams, decided movements and changes were to be expected in the slopes of the dams on the canal side; wherefore, it was necessary to adopt here an excessive width from the first. The width was fixed at 15 m. (49.2 ft.), and the distance between the dams and the axis of the canal was so chosen that the former gave a firm footing for the works on the tops of the banks and the adjoining dikes.

In the section from 7.6 to 13.2 km. the material for the sand dams was more difficult to obtain, the stratum of bog thinner and drier, and the clay was nearly everywhere much firmer. It was proposed to construct this section without sand dams. At a trial excavation at 11 km., slopes of 2 to 1 were found to be stable to the full depth of the canal. The construction of the canal in the natural soil seemed possible as far as it was feasible to build the dikes on either shore, so that their weight on the subsoil induced no lateral pressures on the canal. To attain this end, the plans were arranged so that, first, the banks of the canal and the adjoining berms [2.5 m. (8.2 ft.) in width] should be as high as possible [at + 19.77 m. (64.85 ft.)], and so that the dikes should have a slope of 6 to 1 on the canal side, in order that their crowns

* All elevations which occur in this article refer to a horizontal plane 20 m. (65.6 ft.) below normal zero.

[at + 22 m. (72.16 ft.)] would set back from the edges of the banks of the canal about 16 m. (52.48 ft.). In the Kuden Lake (7.7 to 8.7 km.) where the subsoil was less favorable, the low (embankment) dikes were to be removed from the axis of the canal 70 m. (229.6 ft.).

With the difficulty above mentioned, which arose from the condition of the soil in the lowlands of the Burg-Kuden Lake, the location of the line to be selected required very careful preliminary investigations. Numerous borings were made, extending over the entire area of the lowlands. These proved that the character of the ground on the contemplated line was more favorable than elsewhere, especially on the western border of the lowlands close by the Geest slope, where the line of the so-called Dahlström project, worked up in the year 1880 by the Regierungs-Baumeister Boden, cut across the lowland district. It is true that the soft strata are there not so thick as at a greater distance from the Geest slope, and that the canal bed would therefore cut into the sandy subsoil. But the latter consists of quicksand. In conducting the borings masses of it were mined out by the water flowing over the edges of the bore holes, and it would be a hazardous undertaking to excavate the lower portion of the canal in this material. The necessary dams to be erected on the adopted line will, it is true, be quite costly; but the cost of these can be estimated, and the result is certain. The necessary cost of excavating the bed of the canal in the quicksand could not be predicted, and no assurance could be given that the depth called for in the plans could be reached in this soil by the customary means—dry excavation or dredging under water.

Economic considerations led to a decision in favor of the line adopted. As the plan shows, this line lies in the immediate neighborhood of the district boundary which is, at the same time, the township boundary. It was therefore advisable to buy up the lands lying between the canal and the district boundary, which are cut off by the canal from the township to which they belong, and which are thus inaccessible to their former owners, for spoil banks for which purpose they are well adapted by their position. This also did away with the necessity of arranging for their subsequent economical management, of constructing new roads across the canal, ferries and drainage improvements. Any line lying farther west would have made necessary to continue the profitable utilization of the land so cut off, the estab-

lishment of two ferries, one for Buchholz, and one for Kuden, as well as the construction of parallel roads on the southeast side of the canal, and, in spite of this, the profitable use of the tract so cut off would have been extremely difficult for the former owners. Besides the cost of ferries and roads, and for various new drainage improvements, which would have become necessary in consequence of the division, heavy damages would have been incurred for increasing the difficulty of farming. By the use of the projected line, all these expenses are avoided.

The Execution of the Work in the Section from 7.6 *to* 13.2 *Km.* (see Plates II and III, Figs. 3 to 8).—The work in the section 7.6 to 13.2 km. began by depositing the clay obtained by dry excavation, between 5.8 and 6.6 km., for the formation of dikes as far as 7.6 km. and beyond. But the nearer this construction of dikes approached the Kuden Lake, the deeper they settled into the ground, producing a bulging to the right and left, so that with further excavation slides were to be expected. As, moreover, there was not sufficient material suitable for dikes, the conclusion was forced upon them that it would be necessary to abandon the idea of completing the work without an artificial strengthening of the banks by sand dams. Farther along, in the section from 10 to 12 km., where the upper strata of the soil were removed by dry excavations from the trench and placed to one side, similar experiences were undergone. Only at a few points was the upper stratum firm enough to support the dike without the lateral pressure becoming evident (see the cross-section at 11 km., Fig. 7). Elsewhere, even a slight pressure universally produced motion. It was suggested to remove the bulging slopes of bog for a width of about 3 m. (9.84 ft.) and in their place to substitute a protection of clay of equal amount, which should extend down to the contiguous stratum of clay beneath the submerged embankment. But this was given up because it was feared that the water contained in the soil adjacent, being prevented from escaping by the clay protection, would induce sliding under the pressure of the dike. It was therefore decided to construct such a protection of sand. For this purpose the track, which had already been laid for the filling in of the sand dams from 13.2 to 18 km. with material from the cut in the canal reaching to 26.2 km., had to be prolonged for the whole length of the section in question. This was a difficult piece of work

on account of the slight supporting power of the top stratum. When they began to fill in with sand, the clay was found to be so soft, with the exception of a short stretch, that the sand was soon pressed out of position (see cross-sections, Figs. 6 and 8), and so, in place of the proposed construction of the slopes, real sand dams were built similar in form to those upon the section 13.2 to 18 km., where the construction of such dams had been foreseen from the first. After this experience the attempt to make the slope protections of a given uniform width was abandoned, and the plan of building regular sand dams was adopted.

In the section from 10 to 12 km. the canal was excavated to + 15 m. (49.2 feet), and some of the material for the dikes banked up, before they began the filling in of the sand dams. From 12 to 13 km. and from 10 km. to the Kuden Lake the dams were made in the same way as in the firmer section, 13 to 18 km., where the tracks for extending the work could be relaid upon the top stratum. In relation to the various particulars of the construction the reader is referred to the description (see p. 436, *et seq.*).

For the damming of the Kuden Lake (see Plate II, Fig. 3) a float was employed, by which the work was carried on in the following manner. The material was brought in cars containing 3 cu. m. (105.96 cu. ft.) as far as the tracks allowed, and dumped; then, when the train had gone away, it was taken up in buckets with a capacity of 0.5 cu. m. (17.66 cu. ft.) and dumped on the platform between the end of the dam and the float, to both right and left. Eight or ten buckets were employed for this, which, after being emptied, were taken to the siding between the track and the windlass. After all the cars were emptied, they were taken back to the place for loading sand, and the operation was repeated in the manner described. As soon as the extension of the sand dam had a sufficient power of support, the float with the windlass was drawn forward, the track for the large cars extended, and then the operation was repeated.

We have still to mention a peculiar task with which the commissioners had to occupy themselves before they could begin constructing the dams. The cultivation of the plains, which are drained by removing the peat to + 18.00 m. (59.4 ft.) and then pumping, is accomplished by excavating the fertile calciferous clay (which is usually found in large deposits under the bog and sedge-peat strata) to a depth of

10.5 m. (34.44 ft.) and spread in thin layers over the meadows. For this purpose, peculiar contrivances are used for excavating, called *ketscher*, by the use of which ditches of corresponding depth are cut, about 40 cm. (1.31 ft.) wide, with vertical walls, are formed. These cross the land to be diked here and there, and had to be barricaded where not sufficiently protected by deposits, or where compression by the canal dikes was definitely not to be expected, in order to prevent the water in the canal above from breaking through into the lowlands. The ditch in the diked land to be built over was filled up with sand according to its depth for a length of from 6 m. (19.68 ft.) to 8 m. (26.24 ft.), and through this fill, at right angles to the length of the ditch, sheet piling was placed 2 to 3 m. across and 8 to 10 cm. thick, which should reach to at least 1 m. below the bed of the ditch. In Plate II, Fig. 4, the process is represented. Investigations and observations of the ditches so crossed have shown that the desired result has been attained.

Sand Dams on the Section from 13.2 *to* 18.0 *Km.* (see Plate V, Figs. 11 and 12).—The process of construction differed according as the sand dam was to be built on a firm bed or on one which was soft.

With a firm bed (Fig. 11) the experiment of removing the tough morass cover under the sand dam proved judicious, as, by allowing it to remain and simply loosening it laterally (by narrow trenches), there would usually occur in the operation of filling in the sand a separation of this "mattress" into two parts, preventing the lateral spreading of the sand by assuming an oblique or vertical position. Furthermore, it seemed advisable, in order to effect the most favorable formation of the mass of sand, to give the sand on the edge nearest the canal as deep a bearing as possible. The course pursued in filling in the sand developed as follows in the case of a firm cover of bog.

(*a*) Excavation on the site of the proposed sand dam to a depth of 0.5–1.0 m. (1.64–3.28 ft.), and a width of 12–15 m. (39.36–49.20 ft.).

(*b*) Filling in this trench with sand brought from the completed dam in the rear by means of hand dump cars.

(*c*) Extension of the main track for transporting material [of 0.90 m. (2.95 ft.) gauge] on the edge of the fill next the canal.

(*d*) Dumping the cars loaded with earth [capacity, 3 cu. m. (105.96 cu. ft.)] near this edge until the resulting banks of sand did not sink appreciably.

(*e*) Widening this narrow body of sand on the outer side until the desired width was attained.

For about 800 m. (2 624 ft.) in length of the canal between 16 and 17 km., where the surface of the bog was quite soft and inundated, the preceding excavation was not necessary; but, on the other hand, the preliminary use of rammed piles became necessary. The method of procedure was as follows (see Fig. 12).

(*a*) Driving in light pile-piers in the center of the sand dam penetrating the firm subsoil, and carrying on them a track of 0.6 m. (1.97 ft.) and 0.9 m. (2.95 ft.) gauge (with a common rail).

(*b*) Piling up sand around the piles at night with hand dump cars [of 0.6 m. (1.97 ft.) gauge] from the finished section of the dam behind, in order to enclose the heads of the piles and prevent the pressing in of the bog near the end of the sand dam.

(*c*) Carrying forward the true sand dam with cars of 3 cu. m. (109.96 cu. ft.) capacity, and 0.9 m. (2.95 ft.) gauge in such a manner that the cars unloaded at the end of the dam should be pushed forward on the track mentioned in *a*, which extends over the end of the dam. For a short distance from the end of the sand dam the track should be supported by ways on cross-ties, floats of oil barrels, etc., in place of the piles, which are gradually all broken off by the forward motion of the dam.

(*d*) Widening the dam to its whole width.

By this procedure, especially by the pushing forward of the empty cars on the track laid on piles, the rapid unloading of trains (of 10 to 15 cars), and therefore the rapid progress in the construction of the dam was accomplished.

Results of Filling in with Sand (see Plates III and IV, Figs. 7 to 10).—So far as it could be effected by details in the course of the work, it was attempted to give the sand dams equal slopes on both sides, and where the subsoil was of uniform quality this was successful (see cross-sections, Fig. 10). Exceptions occurred where firm layers of clay with sloping surfaces occurred, in the subsoil, or where (by water courses) there were old and solid dikes on the surface, or where at the beginning of the fill the canal had been partly excavated—where the site of the dike was somewhat overloaded. Under these circumstances a widening of the underlying sand took place towards the outside, or towards the inside of the canal, which, with an increase

pressure upon it, was simply widened without being forced deeper in. The desired forms were established by numerous borings (see cross-sections, Figs. 5 to 10).

The ground displaced by the embankment rose on either side forming irregular upheavals, which frequently extended across the entire space between the dams on the two sides (see cross-sections, Plate IV, Figs. 9 and 10), and rose to 3.20 m. (10.50 ft.) above the original surface, when, after a period of rest, they settled again somewhat. The quantity of sand delivered was estimated by the contents of the transport cars, or by the volume of the unheaved mass within the canal (the volume above of the original estimate) by repeated measurements of the cross-section. The quantity of sand used from 8.2 to 13.2 km., was 590 000 cu. m. (20 838 800 cu. ft.), and the volume of the upheavals resulting therefrom in the canal was 250 000 cu. m. (8 830 000 cu. ft.). From 13.2 to 18 km., 1 216 000 cu. m. (42 949 120 cu. ft.) of sand were used, and the upheaved volume was here 223 000 cu. m. (7 876 360 cu. ft.). The less fortunate condition in the lower section probably resulted from the fact that in consequence of the deeper preliminary excavation for the canal, between 10 and 12 km., the mass of sand settled more towards it than to the outer side; that the ground here, being firmer, was less compressed, and that in the upper section a large amount of sand was used whose excess resulted in no further bulging.

Excavation of the Canal.—According to the specifications, the contractors could only begin excavating between the sand dams after six months of settlement; in reality this time had to be shortened in some sections, as otherwise the excavators would have lain idle. The canal was first excavated in dry material to the banquettes under water with carts, and the material taken out was used partly in building dikes, and partly laid on depositing beds, being used there in the formation of enclosing dikes. In the section from 7.0 to 12.4 km., one large and one small bucket-chain excavator and one pump excavator were used. The latter discharged the mud on the depositing beds through a pipe, while the material raised by the bucket-chain excavator was carried to its destination by two elevators. In the section from 14.8 to 18.0 km., a bucket-chain excavator with a Vering floating apparatus, and temporarily a second bucket-chain excavator with an elevator, have been in use since October, 1891. The section from 12.4 to 14.8 km. as yet remains dry.

Although the bed of the canal has been reached in many places, there have been no large slides in the sand dams, except in one instance—at 14.8 km.—where the borings for the sand dams indicated such a result as probable.

Location of the Dikes.—In the section 13.2 to 18.0 km. but little has been done to the dikes; where the depositing beds are by the canal, as is here generally the case, the masses of earth moved by the apparatus of the excavators are deposited immediately behind the sand dams.

In the lower section 7.6 to 13.2 km., where the flat-dike profile is retained in spite of the sand dams, the dikes are raised to + 20.0 m. (65.60 ft.) with bog soil, to level up the inequalities of the dike lands; marsh land, where over + 20.0 m. (65.60 ft.), is generally leveled off to this height. In this way a level base is formed for the dikes which are to be built of clay, and arranged in such a way that the ground is uniformly loaded. Further building up is performed mainly with an elevator, with a trough having a steep inclination (see Plate V, Fig. 13), which receives the necessary material from a bucket-chain excavator. The steep inclination of the trough renders it possible to handle the material without the assistance of water, in which way the softening of the dike land is avoided. By the constant moving to and fro of the elevator, a gradual and uniform loading of the dike land is accomplished, and it is hoped that in this way slides and uneven settlements will be avoided as much as possible.

PART III.

The East Sea Locks at Holtenau.

I. *General Considerations.*—The East Sea locks at Holtenau form a portion of the lock and harbor works which are being constructed in the vicinity of Kiel, at the eastern terminus of the North and East Sea Canal. They cut off the canal from the harbor of Kiel, and also from the East Sea. Plate VI (Fig. 1) gives a view of the entire harbor works, yet incomplete, the most important features of which are the double locks.

II. *Description of the East Sea Locks.—a. Object of the East Sea Locks.*—The North and East Sea Canal forms, as it were, a strait, whose ordinary surface corresponds to the mean level of the East Sea. As the mean water of the East Sea is at about the same elevation as the mean water of the Elbe at Brunsbüttel, the locks at both ends might be dis-

pensed with were it not that both the East Sea and the Elbe are subject to considerable variations in their water-surface.

In the East Sea, it is true, there are no great changes in the tides, but the maximum and minimum water-levels differ from the mean level by from 3.17 to 2.09 m. (10.40 to 6.85 ft.). It is proposed to leave the gates of the East Sea locks open, with the frequent slight changes of level, so that vessels may pass through without delay. Only when the East Sea rises 0.50 m. (1.64 ft.) above mean water, or falls 0.50 m. (1.64 ft.) below it, will the gates be closed, and then all vessels must be regularly "locked" through. In establishing the depth of the canal the occasional opening of the locks was taken into consideration, the depth being made greater by 0.50 m. (1.64 ft.), *i. e.*, it was increased from the 8.50 m. (27.88 ft.), necessary for the largest war vessels, to 9 m. (29.52 ft.) at mean water-level of the canal (East Sea).

b. General Arrangement and Principal Dimensions.—As one lock would not suffice when there is much traffic, two locks are built beside each other. Another advantage is that the canal need not be closed when one lock is being repaired, or is otherwise rendered useless.

The locks had to be provided with double pairs of gates, on the outside as well as inside, as the outside water-surface of the East Sea is sometimes higher and sometimes lower than that in the canal. The gates in the interior of the lock are not used for locking. They are only used when open passage is closed at mean water in the East Sea. It has been ascertained by calculation that every time the water outside rises or falls rapidly, considerable currents are formed in the canal and locks which, under some circumstances, attain a velocity of 1 m. per second. The calculations show, moreover, that with these comparatively swift currents it would be very difficult and dangerous to close the heavy gates. In order to shut off the current with safety, special "shutting" gates were made, the details of which are described further on. As soon as the shutting gates have accomplished their purpose, *i. e.*, have brought the water in the locks to rest, the regular gates are closed and the shutting gates returned to their recesses, where they remain until it is again necessary to close the locks against the current.

In order to drain the locks dry, rabbets are placed at both ends, into which pontoon gates can be inserted.

The available length of the locks is 150 m. (492 ft.), the length of

the structure being about 217 m. (711.76 ft.), the width between gate piers and in the chambers 25 m. (82 ft.), and the depth of the mitre-sill below mean water 9.57 m. (31.39 ft.). In order to protect the canal and the lower land from the high water, the heads of the locks are built 4 m. (13.12 ft.) above mean water in the East Sea.

Approaches 200 m. (656 ft.) and 100 m. (328 ft.), respectively, adjoin the locks in the outer and inner harbors.

c. Description in Detail.—1. *Ground to be Built Upon.*—The strata uncovered during constructions showed deep intersections by cross-veins which are ascribed to a damming up during the two ice periods in prehistoric times in Schleswig-Holstein. In general a stratum of yellow clay 5 m. (16.40 ft.) thick was found under the vegetable mould, below that usually a stratum of sand which, in turn, lay above a thick bed of blue clay or potter's clay, and, breaking through this in places, a layer of coarser sand is found which contains a quantity of water under heavy pressure. At the northeast wing of the locks, at a depth of 13 m. (42.64 ft.) below the surface of the meadow, a boggy material was found overlying an unstable subsoil of blue clay and potter's clay.

2. *Concrete and Masonry.*—The foundation of the locks consists of a a layer of concrete 2.5 to 3.5 m. (8.20 to 11.48 ft.) thick, enclosed by sheet piling. Where possible, as in the lock chambers, the bed of concrete was given the form of an inverted arch (Plate VII, Fig. 4). At the three heads the form was that of a horizontal rectangular girder (Plate VII, Fig. 3), whose under surface was raised at the side walls in order to economize in the length of sheet piling and to give the latter a firm bearing during the time of deepest excavation and of laying concrete.

The concrete on the bed has three conduits (Plate VII, Fig. 5) diagonal to the axis of the lock, which are curved and enclosed with plates of lead on account of its greater imperviousness to water. The object of these conduits is to convey pipes for hydraulic pressure and heat from a central station lying south of the locks to the machine chamber of the three lock walls. They are also to be used for telegraph and telephone wires.

The masonry above shows on the inside the great culverts with which the locks are filled and emptied. The position of the culverts may be seen in the ground plan (Plate VIII, Fig. 6), and in the cross-sections (Plate VII, Figs. 3 and 4) of the mouth, in the longitudinal sec-

tion (Plate VII, Fig. 5). In order to obviate the formation of strong currents and eddies in filling the locks, the water does not enter the lock chamber in an unbroken stream from each culvert, but through 12 smaller apertures whose combined cross-section is greater than that of the culvert (12 : 7.6 sq. m., or 129.12 : 81.78 sq. ft.), which are distributed over the entire length of the chamber.

In the upper part of the lock walls are the extensive machinery chambers whose arrangement may be seen in the ground plan (Plate VIII, Fig. 6), and the cross-sections (Plate VII, Fig. 3), and in the separate plan of the machinery (Plate IX, Figs. 7 and 8). The covering of the machinery chambers could not be effected with freestone.

1. Because the joints occurring between the two latter could, with difficulty, be kept permanently water-tight.

2. Because the roofing would have been very heavy.

3. Because but a limited height was available for it if the deepest machinery chamber were to be built above mean water-level in the canal.

For these reasons a different roofing was required, and after extended experiments the best was found to be a *monier* roof of peculiar cross-section, laid between iron girders, over which a layer of melted asphalt 3 cm. (1.2 ins.) thick is placed.

The upper part of the concrete bed, the design of the mitre-sill, etc., are clearly shown on Plate VII, Figs. 3 and 4, and Plate VIII, Fig. 6.

The wings which were separate from the actual lock walls could, for the most part, be built directly upon the ground, except the north wing, which had to be built on a grillage set on piles, on account of the boggy soil found there.

As to the materials for the walls, the following is noted : The concrete of the lock bed is composed of a mixture of 9 parts of broken stone (granite) and 5 parts of trass mortar. This trass mortar, which is prepared in mills, is composed of 1 part, by volume, of trass, $\frac{2}{3}$ part of fat lime, and 1 part of sand. The mixing of the broken stone and mortar was carried on in the ordinary wooden drums.

The masonry superstructure and the covering of the concrete in the lock and gate chambers, as well as the wings, are built of brick. Granite blocks are used on all projecting parts, or on those which are exposed to injury, as well as where special dimension stones are necessary. With these exceptions the outside walls are covered with clinkers

in rough-cast mortar, which are brought from Sweden and are particularly tough and very regular in form. Rhine basalt lava is used for covering the lock walls, excepting the machinery chambers. In order to save expense large cavities are left in the brick masonry in the interior of the thick walls, which are rammed full of concrete—a mixture of 1 part, by weight, of cement and 8 parts of sand. The common stones used in the backing are prepared in a kiln arranged for the purpose, in which the brick clay excavated in digging the canal is burnt. The mortar used in the superstructure masonry is composed generally of 1 part by weight of cement, ½ part of fat lime, and 4 parts of sand. This mortar is prepared in the same mills which served for the trass mortar. Comparative experiments have shown that the mortar prepared in the mills is very much stronger than that prepared by hand or in a mortar trough, on account of the more intimate mixture of its ingredients.

In the rough-casting and freestone masonry cement-sand mortar was used, mixed in the proportions 1 : 2 and 1 : 1, respectively. In the construction of the locks there was used approximately—

60 000	cu. m.	(2 119 200	cu. ft.)	concrete.
5 500	"	(194 260	"	) freestone masonry.
65 000	"	(2 595 800	"	) brick masonry.
10 000	"	(353 200	"	) rammed concrete.

The brick masonry, including the rough casting, consumed 26 000 000 bricks.

3. *The Gates.*—The lock gates are arranged as mitre gates. A marked difference exists in the manner of constructing the lock gates and the shutting gates, on account of their different uses. The former are divided into those which shut out the flood in the East Sea and those which hold the water at canal level during the ebb. The head gates are designed as ordinary gates, as they have been transformed from wood to iron methods of construction. The low tail gates, on the contrary, were planned as so-called "fish-pond gates," by which a single strong bolt above holds the gate, vertical posts transferring the hydraulic pressure below directly to the stone sills, and above, to the aforesaid bolt. All the gates together with double-lead water-tight sheathing form caissons.

The shutting gates are so arranged that they can be closed by rush-

ing water without danger. For this purpose they have a large aperture which can be closed by sluice gates, which are left open during the closing, in order to leave a large part of the cross-section of the lock free for the passage of water. Only when the gates are entirely shut are the apertures slowly closed. Until then the sluice gates hang by pulley chains in front of the box (which forms the lower part of a gate wing). That they may be more easily operated, they are supplied with cast-iron pulleys, by which they are moved on guide rails. The gates are made chiefly of cast iron ; the sluices of the shutting gates, of oak.

4. *Pontoon Gates, Sluice Gates of the Feeding Culverts enclosed in the Masonry Capstans, Futtock Heads, Stairs and Fenders.*

At both ends of the locks there are rabbets encased in freestone, in which the pontoons may be set, making the locks water-tight from the inner and outer harbors. The pontoons are made in the usual manner, as a kind of vessel of cast iron. The water ballast is provided for in such a way that the pontoons rise and fall automatically.

The feeders of the locks are closed by wooden rolling sluice gates, which are similar to those of the shutting gates.

The locks are provided with a number of capstans, by which vessels may be drawn in and out, as well as other assistance rendered. Futtock heads and stairs also belong to the equipment, the latter leading from the top of the lock walls to the water.

Floating fenders of wood made of several strong pieces of timber bound together prevent vessels from bumping against the masonry walls. This precaution is taken particularly on account of the passage of large war vessels. If it should be necessary to use the full width of the locks, the fenders can be readily disengaged from their chains at any time and removed.

5. *Mechanical Devices Used in the Locks.*—All the movements necessary in operating the locks, with their gates, sluice gates or capstans, are performed by hydraulic power, but the design is such that, if necessary, they may be done by hand.

The hydraulic pressure is brought from a central station located south of the locks to the hydraulic motors, which are placed in the machinery chambers of each lock wall. Three-cylinder rotating machines are used entirely as motors. As a rule, the arrangement is such that several hydraulic motors work on a common axle, in which way,

by a shifting of the wheels, which may be thrown out of gear when necessary, the gates can be opened and shut, the sluice gates and shutting gates can be raised and lowered, or, finally, the capstans driven. By an extremely simple device in the hydraulic-pressure pipes, corresponding parts on both sides of the same lock move simultaneously and with equal rapidity. The same arrangement makes it possible to produce all the movements from the middle wall of the locks. A gang of men on the side walls will be necessary only for supervision and for services occurring seldom and at stated times. Plate IX (Figs. 7 and 8) illustrates exactly the arrangement of the whole and of the separate hydraulic motors.

It remains to be said that the corresponding shutting gates of both locks perform their motions simultaneously ; that all gates are turned by transferring the motive power by strong racks and pinions ; that the capstans at the ends of the locks have each an especial motor, and that the pressure of the water is 50 atmospheres. All the machinery chambers and passage-ways can be heated by steam by means of steam pipes connected with the boiler in the central power station.

6. *The Approaches.*—The timber approaches provided in the inner and outer harbors in connection with the locks serve as guides for in-going and outgoing vessels. They consist of timber floats which rest against huge clusters of piles, which are able to withstand the shocks from moving vessels.

III. *Description of the Execution of the Work.*—1. *Earthwork.*—On January 5th, 1889, the construction of the East Sea Locks was begun by excavating to a depth of + 13.77, *i. e.*, to 6 m. (19.68 ft.) below mean water of the East Sea. This excavation was surrounded by embankments, which insured protection from the storm floods of the East Sea. The greater part of this work was completed in the course of the same year.

All elevations in this paper refer to a datum plane lying 20 m. (65.60 ft.) below normal zero, or 19.77 m. (64.85 ft.) below mean water of the East Sea.

2. *Control of the Water.*—At the same time experiments were made as to the most successful method of controlling the water for the subsequent work of excavation and laying the foundations. The borings made indicated that a stratum of sand, lying at about ± 0, contained pure water under such a high pressure that it rose in the tubing to

+ 23.77 m. (77.97 ft.) and spouted from the deeper borings as from artesian wells. The ground-water which came up was, as expected, fresh. It is evidently under the pressure of higher and, perhaps, far-distant sources, and flows, as a subterranean stream, to those deep-lying portions of the East Sea where the sand stratum containing the water crops out in the sea. Careful observations of the behavior of the ground-water which were made during construction in the excavations as well as in their surroundings, confirm this hypothesis. With the high pressure of the water in the sand stratum it was feared that, after the excavations in the earth for the concrete bed of the locks were completed, the stratum of clay left between the latter and the water-bearing sand could not withstand the pressure of water from below. It was anticipated that under this pressure strong springs and possibly even larger outflows would occur, and that the pumping of the water would result in a constantly increased loosening of the soil. In order to avoid these evil consequences it was necessary, either to allow the water to rise in the trenches to a considerable height and to lay the concrete under water, or to drain the lower sand stratum in the most judicious manner and thus to lessen the pressure there. The latter method was chosen. A timber shaft was sunk to the lower stratum of sand and furnished with a powerful pump, and in this way it was ascertained that the stratum could be drained and the water pressure diminished by pumping. Three masonry wells, having outer diameters of 5.04 m. (16.53 ft.) and inner diameters of 3.50 m. (11.48 ft.), were sunk to the bottom of the excavation [temporarily at + 13.77 m. (45.17 ft.)], but outside of the sheet piling of the locks. This work was begun on April 12th, 1890, and finished on November 20th of the same year. The lower edges of the wells are at elevations of — 2.13 m. (6.99 ft.), — 0.41 m. (1.34 ft.) and — 0.47 m. (1.54 ft.), respectively.

A fourth well which was originally planned for was afterwards found unnecessary.

The wells were excavated and sunk by the use of compressed air, and filled at the base with coarse sand, gravel and stones, as in a filter. Plate VII, Fig. 4, shows a section of one of the wells which, when the locks are completed, is to be built up to the top of the lock platform and arched over, to be used again in case it is found necessary to drain the locks to lessen the pressure on the lower part of the masonry foundation.

The contracts for the foundations and masonry were awarded in the fall of 1890, and the work begun in January, 1891. The wells were first provided with rotary pumps, and these set to work to drain the lower sand stratum. The control of the ground-water and the desired diminution of pressure were thus attained in the most satisfactory manner. Iron pipes were laid around the excavation at the depth of the sand stratum to observe the intensity of water pressure, and the water stage was taken at regular intervals. During the first months the water sank rapidly, but afterwards more slowly. On the 1st of April, 1892, *i. e.*, after about 1¼ years of constant pumping, the stage in the pipes, which at first reached the average elevation of + 22.02 m., had sunk about 10.25 m. (33.62 ft.) to + 11.77 m. Three boring tubes, which were afterwards driven under the middle wall, indicated a stage, on an average, of less than + 7 m. (22.96 ft.), so that the head of the ground-water in the inside of the excavation was diminished by more than 15 m. (49.20 ft.). There resulted from drawing off the ground-water an excavation almost completely dry, into which the strata of clay allowed no water to pass, and the sand stratum, which was exposed in some places, allowed but a few springs with slight pressure to come in. The water entering by these springs, as well as all water entering the excavation from outside, was conducted to the wells through clay pipes 30 cm. in diameter, laid with open joints in gravel through the entire length of the middle of each wall. Two transverse lines connect these pipes with one another and with the three pump wells, forming a complete network of drains, with minor lateral lines. The cross-sections (Plate VII, Figs. 3 and 4) show the position of the clay pipes.

The three pump wells have cost about 100 000 marks, exclusive of pumps, but this is more than balanced by the greater economy in construction—the lessening of the earthwork and the thickness of the concrete, the substitution of thin partitions (moulds) for the temporary sheet-piling of the middle wall, etc.

3. *Work on Foundations and Laying of Masonry.*—From this point the work was regularly prosecuted without any particular events.

The foundation work was begun on the 19th of March, 1891, by deepening the excavation to + 10.77 m. Next, the concrete and masonry of the lock walls were finished, and, after the latter had entirely settled, the concrete and upper masonry of the lock beds were completed.

After the earthwork was carried down to + 10.77 m., the sheet piling of the side walls was driven. The earth between these rows of sheeting was dug out to the projected depth and the concrete then laid. The earth under the middle wall was removed, and after the moulds provided for enclosing the masses of concrete were torn down, the concrete was laid there also.

Excavation was done partly with a "dry" excavator and partly by hand. The sheet piling was driven mostly with direct-acting drivers; in a few cases with steam pile-drivers. The concrete was laid to its full thickness at once by means of a tipping trough with a light tamping of the surface.

Concrete work on the longitudinal walls was begun July 6th, 1891, and finished November 25th, 1891. In the beginning of September of the same year the concrete first laid had sufficiently hardened to begin the upper masonry.

The winter interrupted all work, which was resumed in March, 1892. The lock walls were nearly completed in the year 1892. It was possible to lay about three-fourths of the concrete of the lock beds during the second half of the year 1892. It is expected that the concrete and masonry will be completed during the year 1893, excepting for a few pieces of supplementary work.

4. *Gates and Mechanical Arrangements.*—The gates which are already begun are expected to be placed in July, 1893. Portions of the machinery were installed in the machinery chambers of the side walls in 1892. The placing of the gates and the installation of the machinery necessary for their operation, as well as for working the sluice gates, the feeder gates and the capstans, are to be finished according to the contract by April 1st, 1894.

5. *The Approaches.*—The completion of the approaches will be prosecuted during the years 1893 to 1895.

6. *Time of Completion.*—It is intended—and without unlooked-for delays it can be accomplished—that the East Sea locks shall be ready for use July 1st, 1894, so that the removal of the coffer-dams can be begun. The opening of the entire canal for commerce will follow in the summer of 1895.

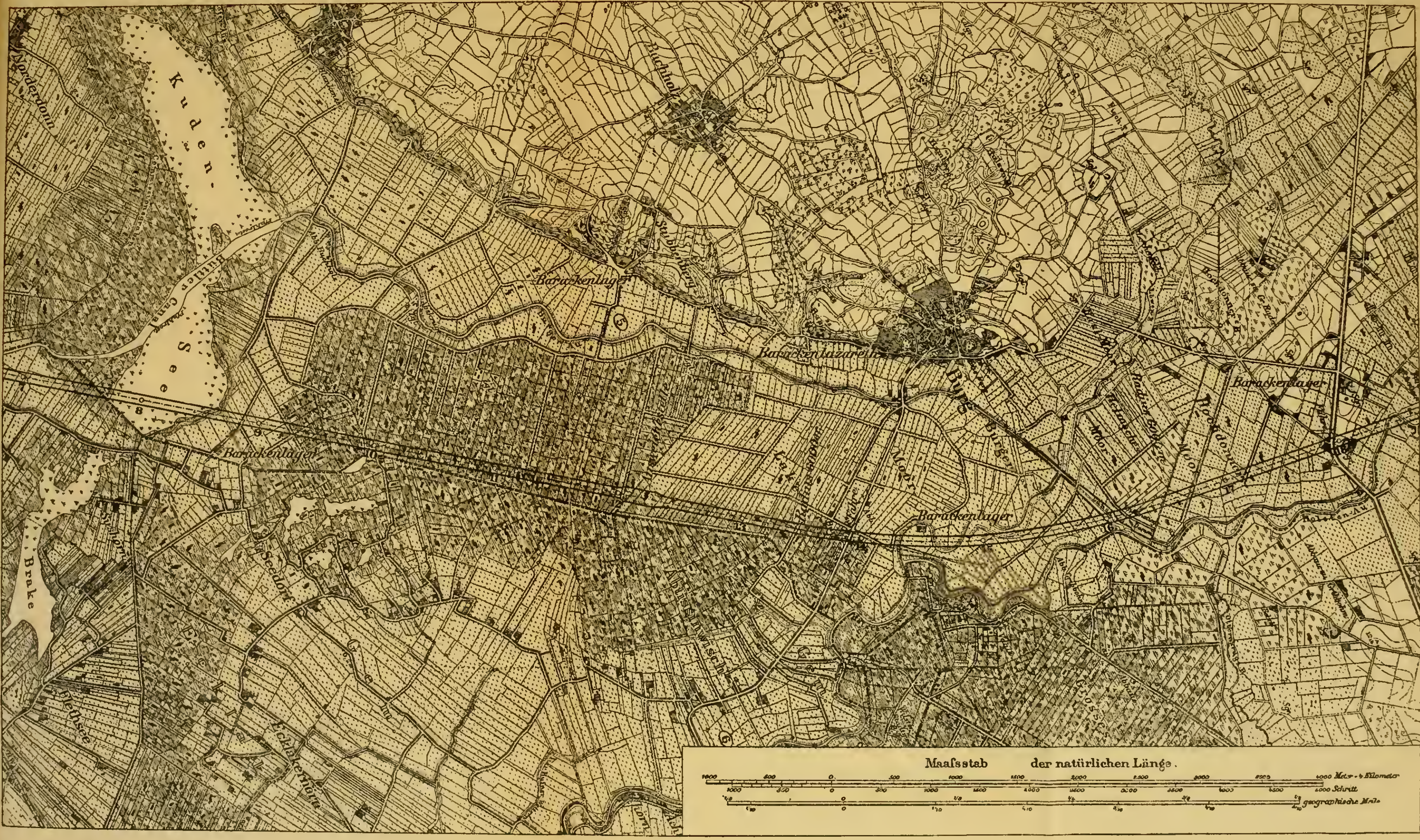

Kuden-See
Buchholz
Barackenlager
Barackenlazareth
Burg
Barackenlager
Barackenlager
Barackenlager
Brake
Maafsstab der natürlichen Länge.

Fig. 2. Geologisches Längenprofil.
(Geological section of the canal-line.)
Km 6,80 - Km 18,00

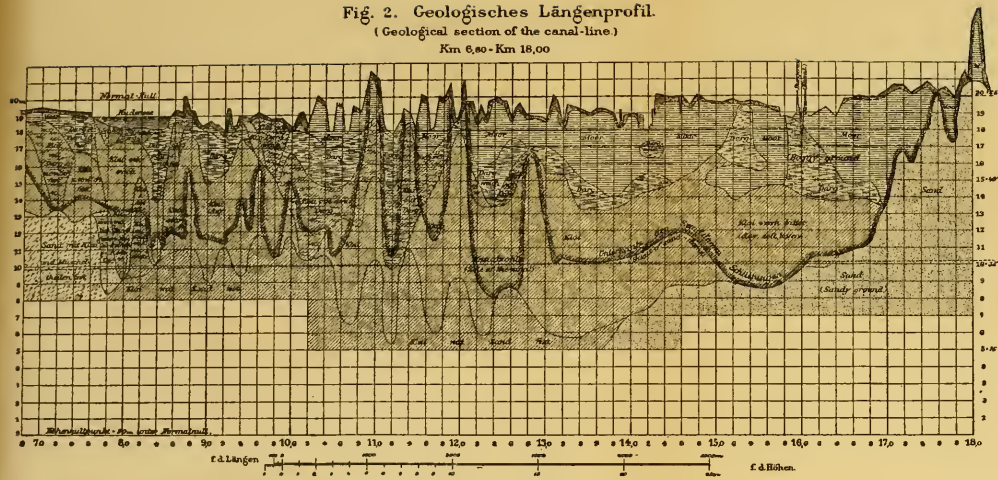


Fig. 3a - c. Schüttung der Sanddämme im Kudensee.
(Casting up of sand-dams in the Kudensee.)
Fig. 3a Lageplan.

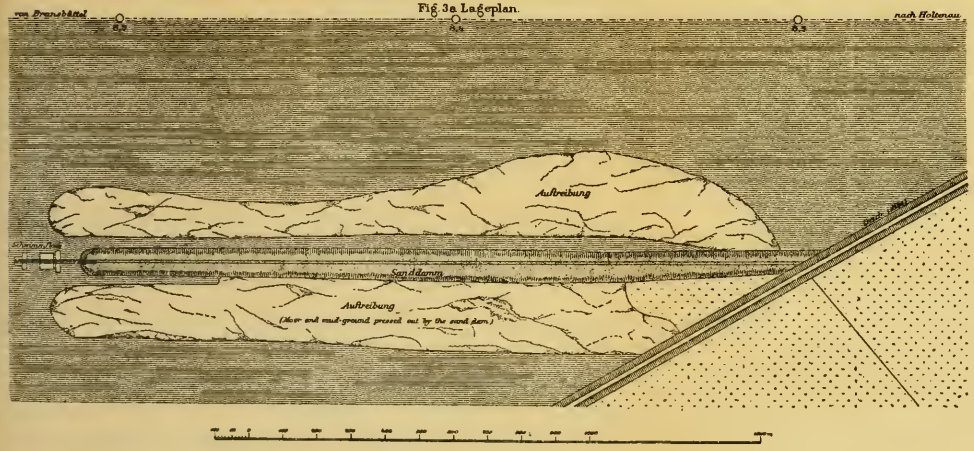


Fig. 4a - d. Verbauung der Ketschergräben
(Filling up the narrow ditches. Depth 20 - 33')

Fig. 4a Längenschnitt durch einem abgeschlossenen Ketschergraben.
Fig. 4b. Grundriss zu Fig. 4a.

Fig. 3b. Längsschnitt durch die Schwimmflossvorrichtung
(Longitudinal section of the float.)

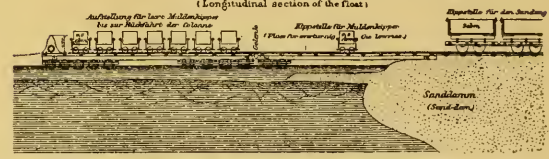
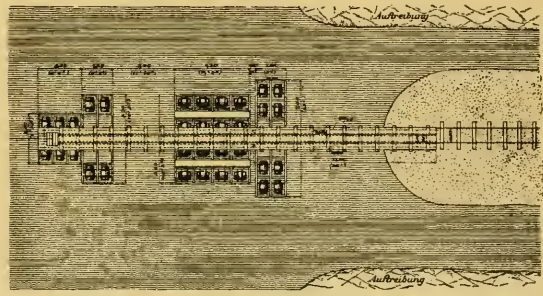


Fig. 3c. Aufsicht auf die Schwimmflossvorrichtung
(Plan of the float.)



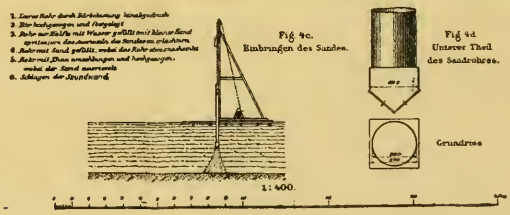
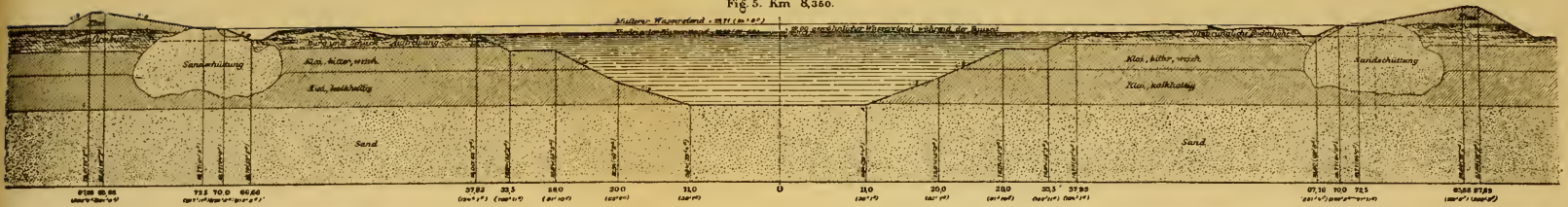


Fig. 5-8. Querprofile.

(Cross - sections.)

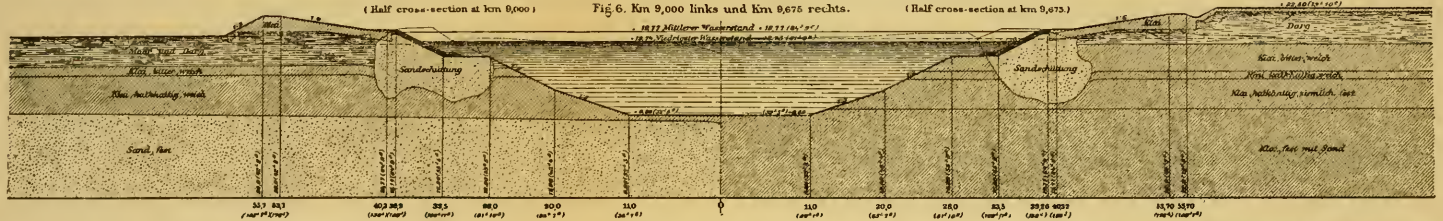
Fig. 5. Km 8,350.



(Half cross-section at km 9,000)

Fig. 6. Km 9,000 links und Km 9,675 rechts.

(Half cross-section at km 9,675.)



(Half cross-section at km 10,500)

Fig. 7. Km 10,500 links und Km 11,000 rechts.

(Half cross-section at km 11,000.)

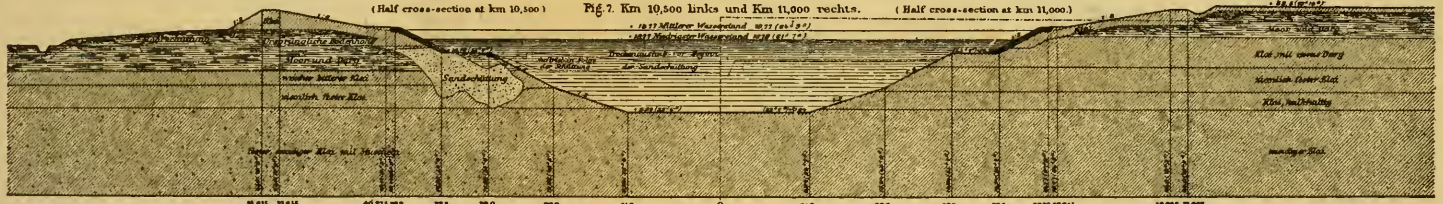
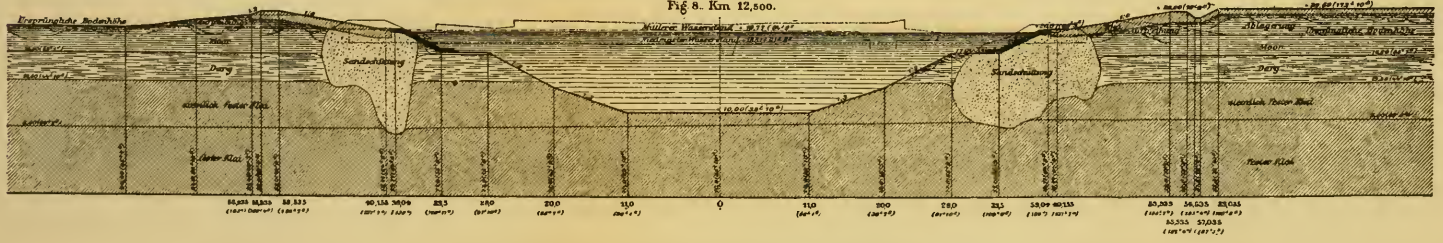
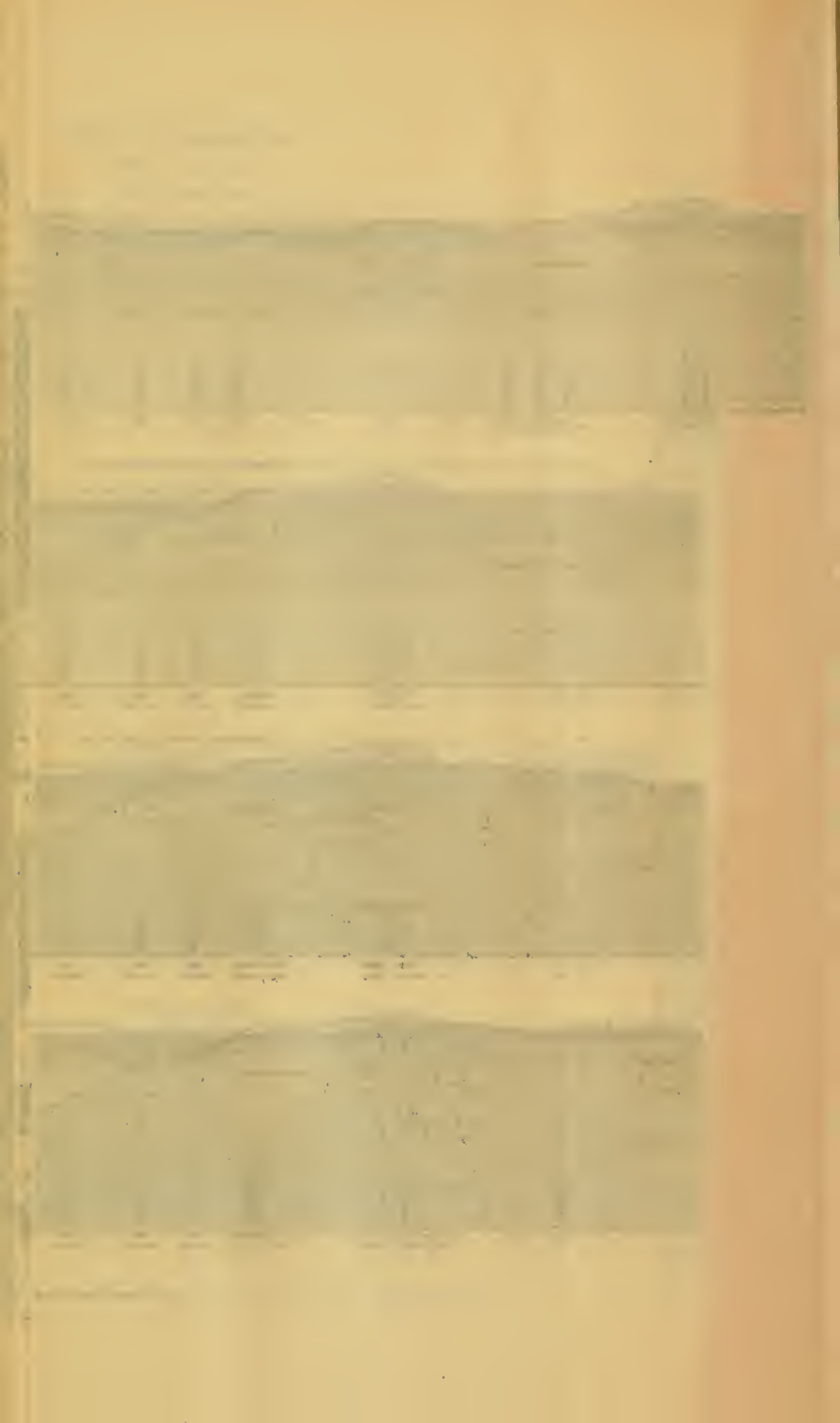


Fig. 8. Km 12,500.





Sanddammschüttungen im Moor.

Km 13,2–19,1.

Fig. 9. Ungewöhnliche Sanddammquerschnitte.

(Cross-sections of irregular sand-dams.)

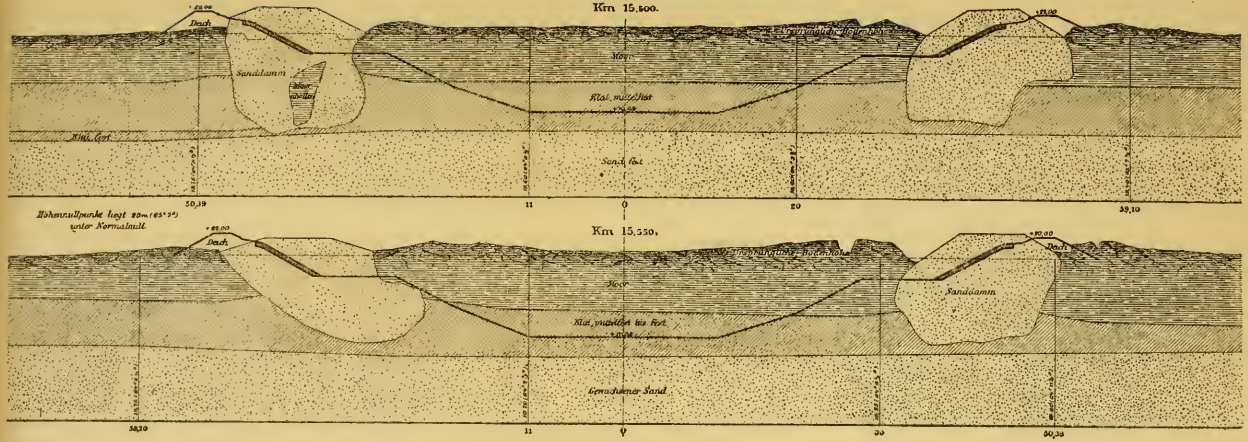
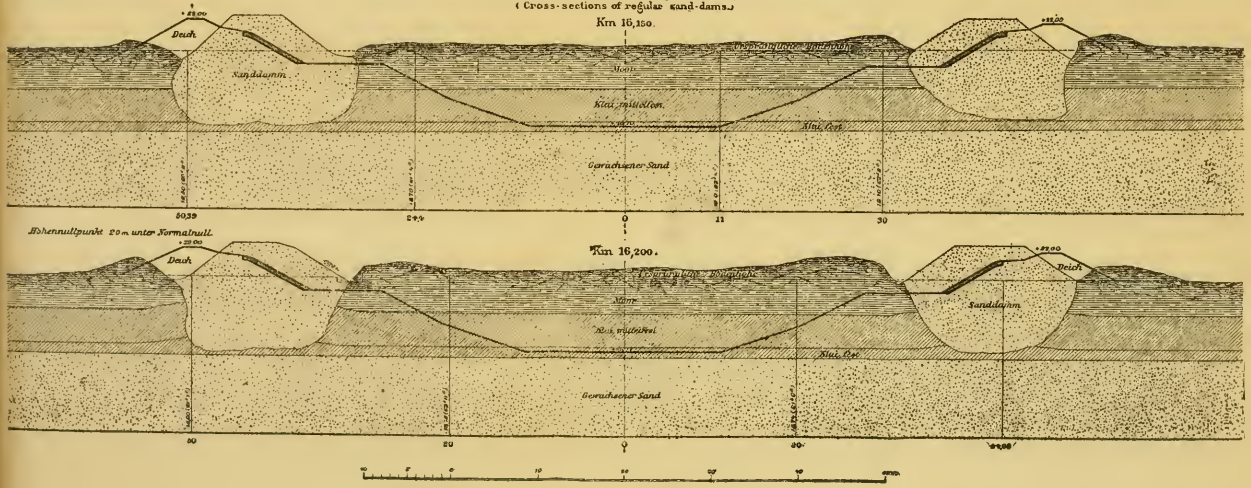


Fig. 10. Normale Sanddammquerschnitte.

(Cross-sections of regular sand-dams.)

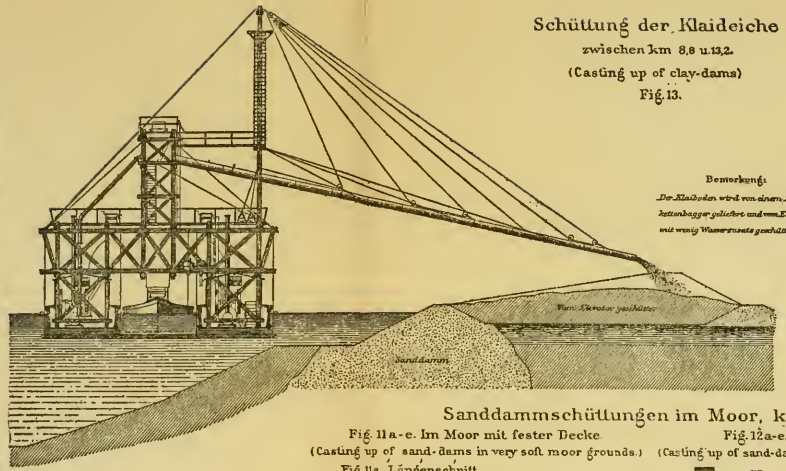


Schüttung der Klaideiche

zwischen km 8,8 u. 13,2.

(Casting up of clay-dams)

Fig. 13.



Bemerkung:
Der Klaiboden wird von einem Eimerkettenbagger geliefert und vom Elevator mit wenig Wassersusatz geschüttet.

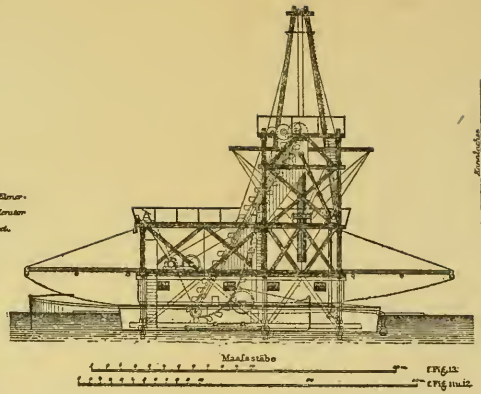


Fig. 11c. Querschnitt A-B.
(cross section A-B.)

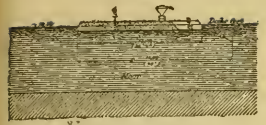


Fig. 11d. Querschnitt C-D.
(cross section C-D.)

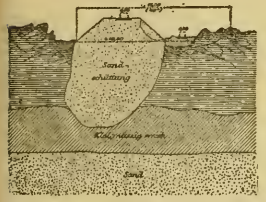


Fig. 11e. Querschnitt E-F des fertigen Sanddammes.
(cross section E-F of finished sand-dam)

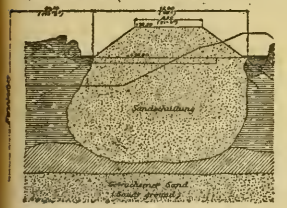


Fig. 12c. Querschnitt A-B.

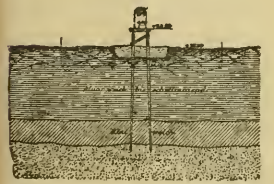


Fig. 12e. Querschnitt E-F des fertigen Sanddammes.

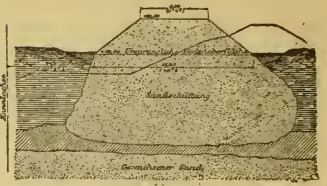
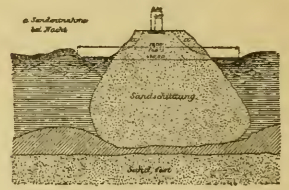


Fig. 12d. Querschnitt C-D.



Sanddammschüttungen im Moor, km 13,1 - 19,1.

Fig. 11a-e. Im Moor mit fester Decke.
(Casting up of sand-dams in very soft moor grounds.)

Fig. 12a-e. In ganz weichem Moor.
(Casting up of sand-dams in moor with settled upper-part.)

Fig. 11a. Längenschnitt.
(Longitudinal section)

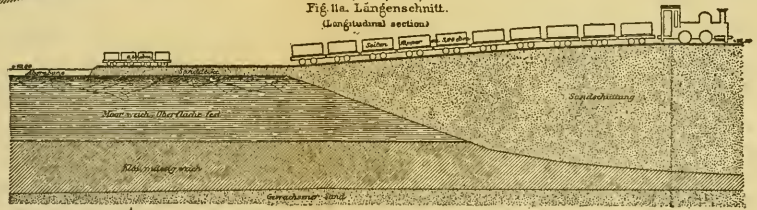


Fig. 11b. Aufsicht.
(Plan.)

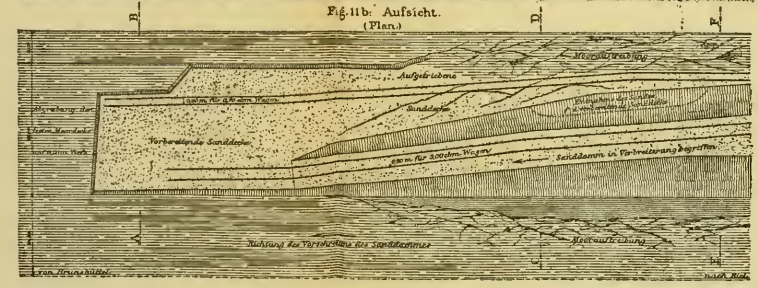


Fig. 12a. Längenschnitt.
(Longitudinal section)

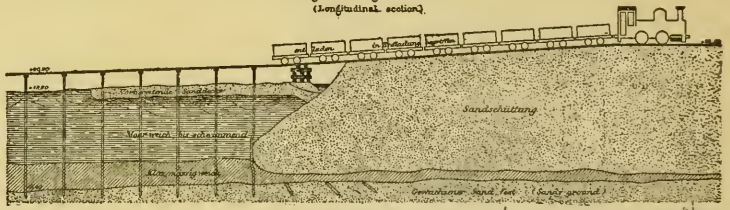


Fig. 12b. Aufsicht.
(Plan.)

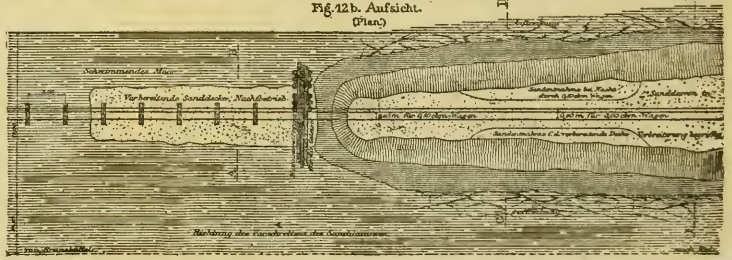


Fig. 1.

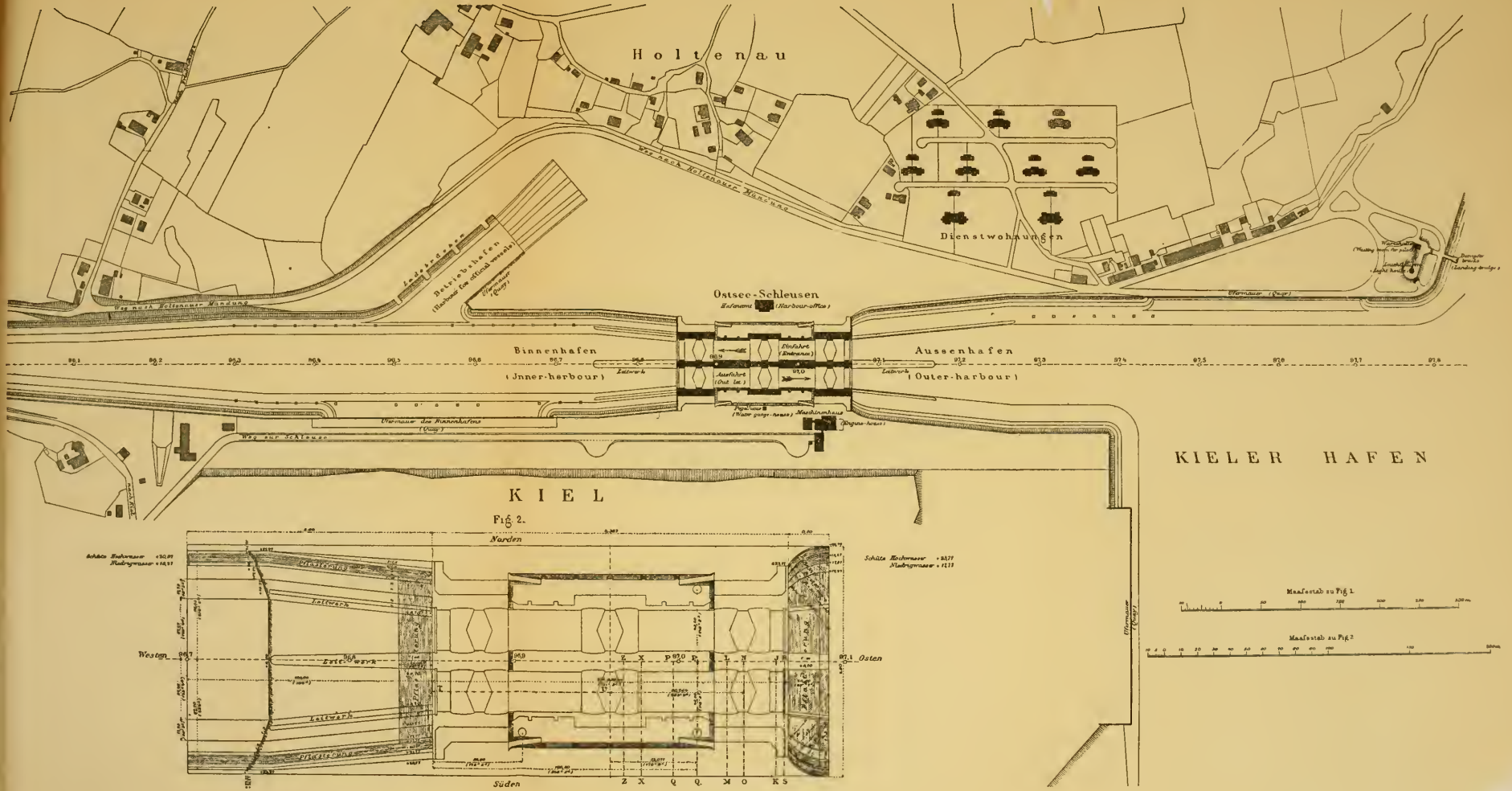

Holtenau
Weg nach Holtenauer Vorhafen
Betriebshafen
(Harbour for official vessels)
Dienstwohnungen
Ostsee-Schleusen
Befahrt (Harbour-office)
Binnenhafen
(Inner-harbour)
Einfahrt (Entrance)
Ausfahrt (Out. let.)
Aussenhafen
(Outer-harbour)
Leitwerk
Pegelhaus (Water gauge-house)
Maschinenhaus (Engine-house)
Weg zur Schleuse
KIEL
KIELER HAFEN

Fig. 2.

Norden
Süden
Westen
Osten
Leitwerk
Ende Westwerk
Pegelhaus
Z X Q Q M O K S

Maafsstab zu Fig. 1.
Maafsstab zu Fig. 2.

Fig. 3. Querschnitt (Section) durch die Schifffahrtsthorkammer (of the lockgate-chamber) durch die Schifffahrtsthorpfeiler u. Drempel (of the lockgates-pillar and sill.)

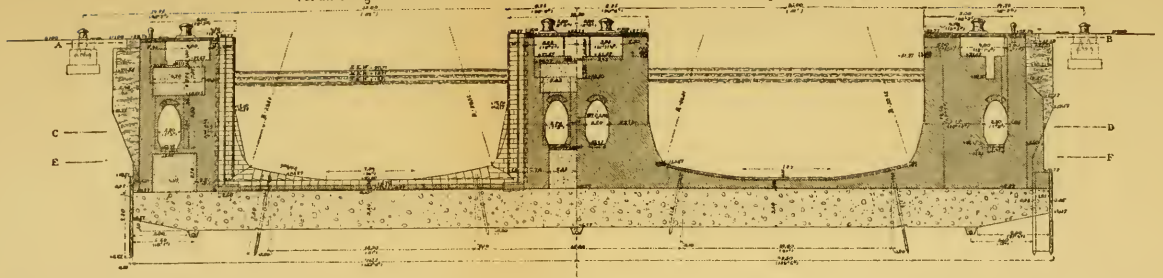


Fig. 4. Querschnitt durch die Schleusenkammer. (Section of the lock-chamber.)

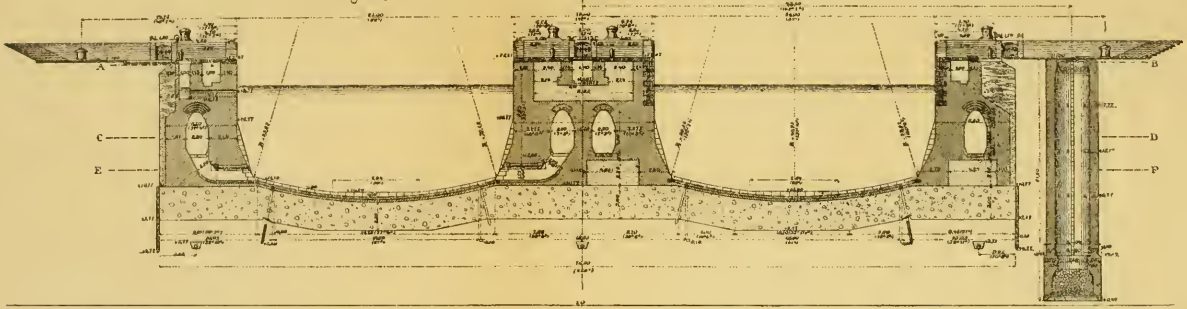
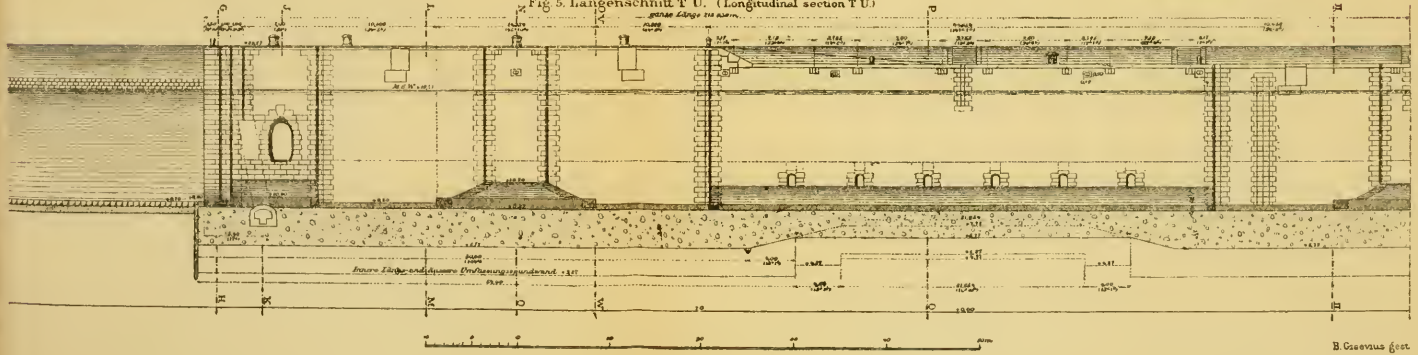


Fig. 5. Längenschnitt T U. (Longitudinal section T U.)



B. Gaevius gest.

Ostsee-Schleusen.

Fig. 6.

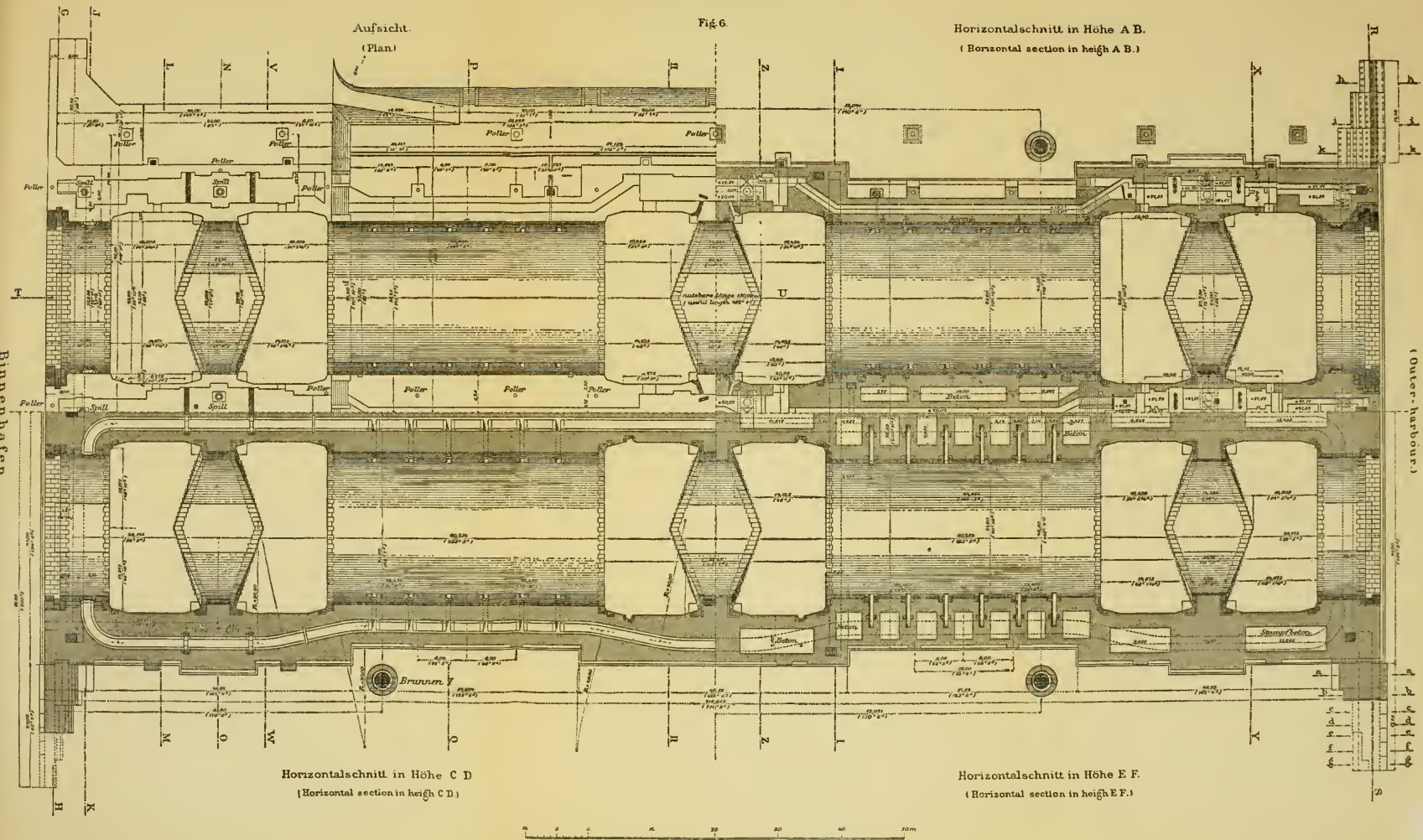
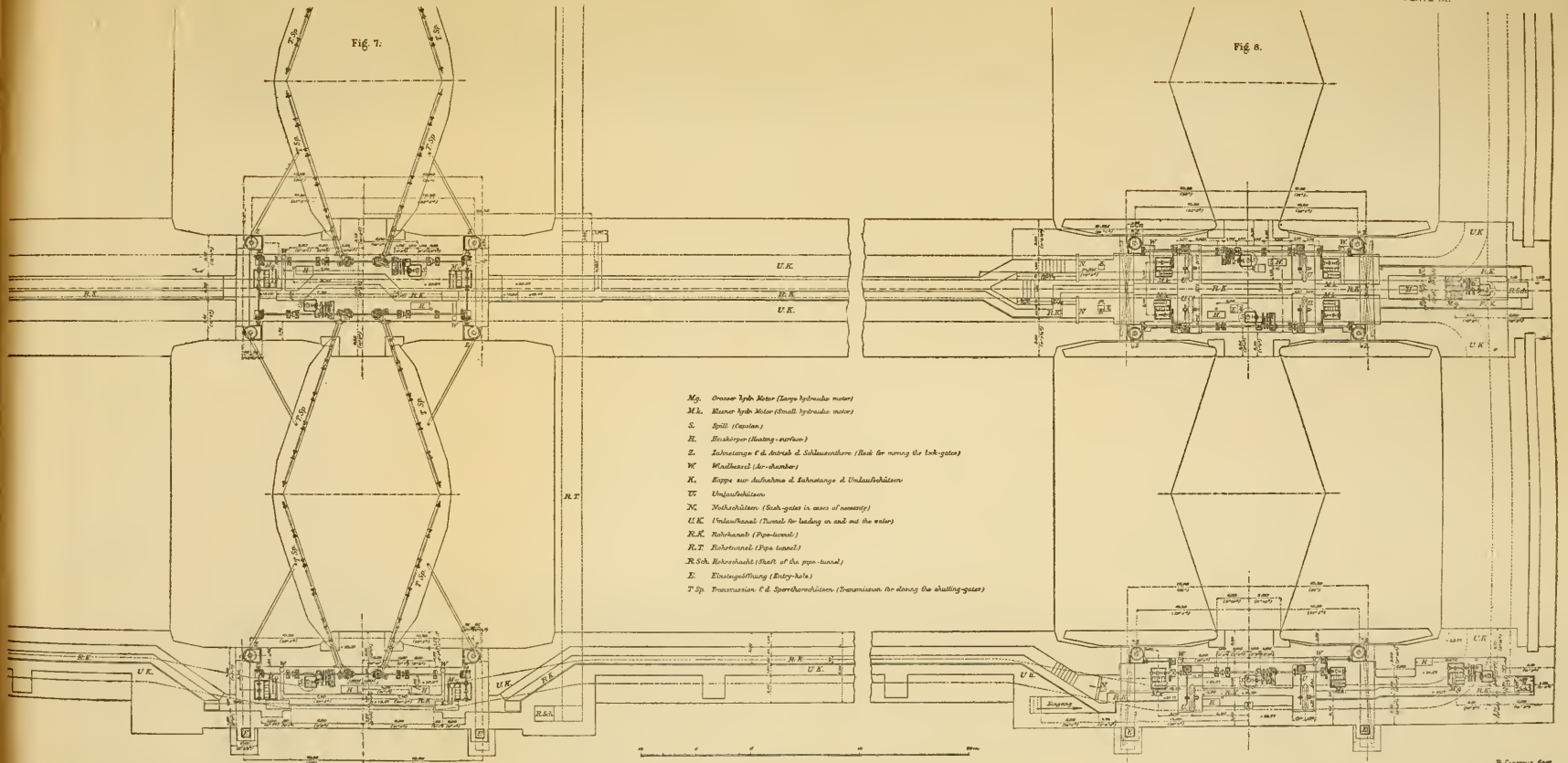


Fig. 7.

Fig. 8.



Mg. Grosser hydr. Motor (Large hydraulic motor)
Mk. Kleiner hydr. Motor (Small hydraulic motor)
S. Spill (Capstan.)
H. Heizkörper (Heating-surface)
Z. Zahnstange f. d. Antrieb d. Schleusenthore (Rack for moving the lock-gates)
W. Windkessel (Air-chamber)
K. Klappe zur Aufnahme d. Zahnstange d. Umlaufschützen
U. Umlaufschützen
N. Nothschützen. (Stop-gates in cases of necessity)
U.K. Umlaufkanal (Tunnel for leading in and out the water)
R.K. Rohrkanal (Pipe-tunnel)
R.T. Rohrtunnel (Pipe tunnel)
R.Sch. Rohrschacht (Shaft of the pipe-tunnel)
E. Einstiegsöffnung (Entry-hole)
T.Sp. Transmission f. d. Sperrbarschützen. (Transmission for closing the shutting-gates)

B Casemus Sc.

7. *Cost of the East Sea Locks.*—The cost of the East Sea locks, exclusive of the approaches, will amount to about 9 000 000 marks.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

655.

(Vol. XXX.—November, 1893.)

THE NEW RAILWAY STATIONS AT DRESDEN.

By C. KÖPCKE, Geheimer Finanzrath, and O. KLETTE, Baurath, in Dresden, Germany.

Prepared for the International Engineering Congress of the Columbian Exposition, 1893.

The first railway in Saxony, and, at the same time, the first considerable railway in Germany, was that between Leipzig and Dresden, commenced in the year 1835, and opened for traffic in full length of 115 km., or 72 miles, in 1839. This railway has its terminus in Dresden-Neustadt, which, exclusively of enlargements, is still at the same place as 54 years ago. The station is situated on the right bank of the Elbe River, as marked by the letter *A* on Plate I.

The terminus of the second railway built in Saxony, from Dresden to Görlitz, which was opened for traffic in 1846, has its place also in Dresden-Neustadt, near to that of the Leipzig station, with which it was early connected by a siding for freight traffic. It is marked in the plan with *B*.

NOTE.—Discussions on all papers presented to the International Engineering Congress will be published simultaneously in the number for December, 1893.

The third line terminating at Dresden is the Saxon-Bohemian Railway to Prague and Vienna, completed in the year 1849 by the Saxon government, after it had been ceded by the company which had undertaken it. The terminus of this line is the Bohemian station, marked in the plan with *C*. A prolongation with a long viaduct and a bridge over the Elbe unites it to the stations *A* and *B* of the older lines in Dresden-Neustadt.

Another company constructed the short Albert-Bahn, named after the present King, then Crown Prince Albert of Saxony, from Dresden to Tharand, finished in 1854, with a terminus at *D* in the plan. It is now only a coal station, the passenger traffic, in consequence of the purchase and extension of the railway by the Government, having been transferred to the Bohemian station *C*.

Finally, in the year 1875, the railway Berlin-Dresden was finished by a company, with a terminus in Dresden-Friedrichstadt at *E* in the plan, and with a connecting track to the Bohemian station.

Thus there are now in Dresden four passenger stations. As to the freight stations, those of the lines ending at the points *A*, *B*, *D* and *E* of the plan are still in their original places, whereas the freight station of the Bohemian line, in the interest of the City of Dresden, wanting space for extension, has been transferred to the point *F*, between *C* and *D*. This new freight station *F*, with an inclination of 1 in 110 at its western end, was fitted also for sorting and shunting purposes, and provided with repair shops.

This multiplicity of railway stations, in a place of (1890) 305 000 inhabitants, lost its reason for existence in the same degree as the railways passed over into the management of the Government. After the acquisition of the lines to Görlitz, Bohemia (as far as Bodenbach) and to Tharand, the Government bought in 1876 the lines of the Leipzig and Dresden Company, and at last, in 1888, the Berlin and Dresden line from Dresden up to the limit of the kingdom. Several projects had formerly been made for a better connection of the Dresden stations, especially the passenger depots. But, after all the railways had become the property of the State, a total transformation of the railway system, including the freight stations, was planned. In addition, the river navigation on the Elbe and its growing importance for Dresden ripened the plan of a large river harbor, which necessarily had to be connected with the railways.

The problem to be solved was therefore:

First.—The erection of a central passenger station; for which the Bohemian station *C* in the plan was chosen.

Second.—The establishment of a central sorting or shunting station, which will be placed at *E* in the plan, by enlarging the Berlin station, near which also the river harbor will be excavated.

Third.—The connection of the passenger stations on the right bank of the Elbe at *B* in the plan, and their transformation into an intermediate station for all through trains of the Leipzig, Berlin and Görlitz lines (the passenger traffic for Berlin being transferred to the Leipzig line).

Fourth.—The combination of the freight stations in Dresden-Neustadt on the ground of the Leipzig station *A*.

Fifth.—The laying of sidings on a part of the freight station *F*, for storing and shunting passenger cars, and making up the passenger trains, and, in connection therewith, the erection of the necessary buildings for a central post-office.

Sixth.—The abolition of the level crossings.

As may be observed by looking at the plan, the passenger and freight traffics will be held entirely separate by passing the freight trains through the whole city without contact with the passenger tracks. All constructions for the passenger traffic are placed on the side towards the town, and those for the freight traffic on the outside. Instead of four shunting yards, as until now in use, between which the freight cars not intended for loading or unloading at the arrival station must be carried to and fro, there will be common sorting and shunting yard combined, with a large repairing establishment, at *E* in the plan. Here all the freight trains, including those from the local freight stations of Dresden, will be received. For this purpose there are planned at *a* 17 entering sidings of 600 m. (2 000 ft.) in length for receiving trains numbering 75 cars. The access to these sidings is nearly independent for all the different railway lines, and four freight trains may enter at the same time. After entering, the engine as well as the trainmen leave the train, and the papers relating to the contents are taken to an office in the immediate neighborhood, where they are separated according to the destinations of the various cars. After the arrival of a train, a shunting engine moves the cars on a double-track railway, laid with an inclination of 1 in 80 to the

group *b*, of five inclined sidings, which converge into an end siding, by which the shunting engine leaves the train, to pull up a new one. The whole sorting station forms an inclined embankment of 17 m. (57 ft.) greatest height, filled up with the material gained from the river harbor, about 1 500 000 cu. m. As the uniform inclination of the sorting sidings is 1 in 100, the freight cars run by gravity, and experience has shown that this inclination is sufficient, even during strong contrary winds. Out of the group of inclined sidings *b*, the cars run either single or in groups, having the same destination, into the sorting sidings, where they are stopped by brake-blocks laid on the rails. There are 30 parallel sorting sidings (*c* in the plan), of which every one receives the cars for a particular direction, be it in Dresden itself or outward on the lines. Now it remains to arrange the cars destined for the intermediate stations in the succession necessary for avoiding all subsequent shunting. This is done by using the so-called gridirons (*d* in the plan), harp-shaped groups of sidings. In the upper gridiron, with eight parallel sidings, eight groups of cars can be formed, and in the second gridiron below the former, with four parallel sidings, every group may be separated into four parts, so that it is possible to divide a train in $4 \times 8 = 32$ different manners, by letting them move in proper succession after one another. For the freight trains starting from Dresden, there are no more than 16 different destinations of the freight cars, and ordinarily only eight different stations come into consideration, so that the arranging of a train according to the succession of the stations can be finished in the first gridiron. Nor is this number exceeded by the branches to the different freight houses, mills and other loading places passed by the local trains in Dresden. After passing the gridirons, the cars run into the departure sidings *e*, where they are coupled. These sidings are 14 in number, and near to them are placed the engine houses, so that after being manned and furnished with the necessary papers from the office, to which they had been taken, the train may leave the station, and this departure can be effected in four different directions at the same time. The cars wanting repairs are moved from group *c* into special sidings, from which they are conveyed to the repairing shops at *f* of the plan.

The length of the sorting yard is 2 500 m. (8 300 ft.); its breadth, including the workshops, is 600 m. (2 000 ft.). There are 51 000 m. (170 000 ft.) of tracks and sidings. As the plan shows, there is still

room for a considerable enlargement of the works now in course of construction.

The freight stations are marked in the plan by the letters *A*, *D*, *F*, *G*, *H* and *I;* they are all in direct connection with the sorting yard, the construction of which was begun in the year 1891, and will probably be opened for traffic in the month of May, 1894.

The sorting by gravity has been practiced many years in Saxony, and a publication on this method of arranging the cars in the "Organ für Eisenbahnwesen," 1871, page 60, has helped to give an impulse to its extension over Germany. In England Mr. Footner has introduced the gridirons, invented by him, on the shunting station at Edgehill, near Liverpool, as is described in the "Working and Management of an English Railway," by George Findlay, London, 1889, under the chapter of "The Shunting and Marshalling of Goods Trains."

The investigations made by a commission of officers of German railways have shown that the arranging by inclined sidings requires no more than half the time and cost of the ordinary shunting by the locomotive, and besides causing less damage to men, to cars and to the goods conveyed, this proceeding needs the least extension of the sorting station. The use of gravity was formerly limited either to separating the cars for the different destinations, or to collect them on the departure siding, one-half of the work—commonly the collecting —being always done on level sidings by the locomotive. The invention of Mr. Footner doubles the profits of the working by gravity, as it unites both the separation and the combination of the cars. The new sorting yard at Dresden will be the first application of Mr. Footner's method in Germany.

The chief advantage of the concentration of all the shunting and marshalling works of a large center of commerce is the quick dispatch of arriving freight trains. Though the movement of cars between some of the local freight stations and the sorting yard will be multiplied, the saving gained by the quick expedition in the transit of cars overbalances that little disadvantage.

At Dresden the number of cars to be shunted daily is about 3 000; but the new yard is large enough for arranging 4 000 cars, and, after the possible extensions, the number of cars to be moved may be still greater. In the neighborhood of the sorting yard there are large tracts of land for building purposes, and especially the area between

the yard and the river harbor, owned by the Government, offers an opportunity for erecting industrial establishments with branch lines.

The establishments for the passenger traffic are with reference to the plan :

First.—The central passenger station *C.*

Second.—The terminal station *L.*

Third.—The passenger stations at Neustadt *B*, at Friedrichstadt *E*, and Wettin Strasse *K*, the latter two serving chiefly for the city and suburban traffic, for which other stopping-places may be erected in the future at *M* and *N*.

The central station *C* has a very suitable position, lying at a distance of only 1 km. from the center of the city. After the open grounds fit for buildings are covered, this station will be situated in the midst of the Altstadt or the earlier part of the city, on the left bank of the river. For the Neustadt, on the right bank, the station *B* will be well accommodated to the passenger traffic.

At the central station *C*, the tracks and platforms will be built on two levels, differing by 4.4 m. (14 ft. 5 ins.). For the east and west line, Görlitz-Dresden-Chemnitz, the central station will be used as a "pocket-station," ending before Prager Strasse, the first street crossed at present on the level by railway tracks. Now, as the tracks will be cut off before reaching that street, it is not necessary to elevate them; but the elevation is required for the tracks passing beyond the station, in order to abolish the level crossings of Prager Strasse, as well as of a number of other streets on the southern side of it in the direction to Bodenbach and Prague. Thus to the passengers using the six pocket-station tracks—about one-half of all—the mounting of stairs will be spared.

The central station will be connected by sidings with the terminal station *L*, at which all passenger trains, excepting the through trains, are to begin and end their movements. Therefore, this latter station will afford ample conveniences for the making-up of trains and for sheltering of engines; also, for the despatch of parcel goods and of the whole railway business of the post-office.

The passenger stations *K*, *B* and *E* will only have intermediate traffic in connection with the central station. The tracks and sidings of these stations are either above (*B* and *K*) or below (*E*) the public streets, and are therefore accessible only by stairs.

The following is a synopsis of the areas and length of tracks and sidings of the new stations:

	Square meters.	Meters of track.
1. Central station *C*	180 000	23 000
2. Terminal station *L*	200 000	23 000
3. Intermediate station, Wettin Strasse *K*	25 000	1 500
4. Intermediate station, Friedrichstadt *E*	15 000	1 500
5. Station Neustadt *B*	130 000	18 000
Total for passenger traffic	550 000	67 000

	Square meters.	Meters of track.
6. The sorting and shunting yard *H* has an area of	750 000	98 000
7. Freight station, Altstadt, *F*	135 000	15 000
8. Coal station *D*	95 000	11 000
9. Freight station, Neustadt, *A* and *I*	310 000	31 000
10. Station at the harbor *G*	200 000	19 500
11. Old quays on the river at *O* and *P* of the plan	90 000	8 500
Total for freight traffic	1 580 000	183 000

The sum of the areas adapted to the railway traffic of Dresden will, therefore, be 158 + 55 = 213 hectares (about 500 acres), and the length of tracks, about 250 km. or 155 miles. As the area of the city is 2 858 hectares, the railways will occupy more than 7% of it.

VOL. XXX, No. 655.
TRANS. AM. SOC. CIV. ENGRS.
KÖPCKE ON RAILWAY STATIONS AT DRESDEN.
PLATE 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

656.

(Vol. XXX.—December, 1893.)

INLAND TRANSPORTATION.—DISCUSSION ON PAPER No. 605.*

By C. W. RAYMOND, EDWARD P. NORTH and F. A. MAHAN.

C. W. RAYMOND, M. Am. Soc. C. E.†—Captain Mahan's paper raises many important questions of national and commercial policy which it is impossible to discuss adequately on this occasion. While fully recognizing the ability and literary skill with which he has treated the subject of inland transportation, I am compelled to dissent from many of his views.

The main foundation of his argument in favor of the construction and maintenance of artificial water-ways is the assumption that they furnish cheaper transportation than railways for heavy and bulky materials. This he endeavors to establish from a comparison of freight rates for grain on various railways and water lines, showing, he thinks, a decided economy in favor of the water transportation. For example, the average freight rates per bushel of wheat from Chicago to New York in 1891 were, by lake and rail, 8.53 cents, and by lake and canal, 5.96 cents, from which it is inferred that there is for the latter route a

* "Inland Transportation." By Capt. F. A. Mahan, Corps of Engrs., U. S. A. *Transactions*, Vol. XXIX, page 97.

† Major, Corps of Engrs., U. S. A.

saving in the cost of transportation of 2.57 cents per bushel. Supposing all the grain moved from Chicago to New York in 1891 to have been carried over the water-route, the writer considers that there would have been a saving of $3 208 500 in the total cost of transportation.

The fallacy in this method of comparison lies in the fact that it confuses the freight rate with the net cost of transportation, two very different things. The freight rate is simply the price paid for transportation. The net cost of transportation includes two principal elements—the toll for use of way, and the cost of transport proper. The toll for use of way covers the interest on the cost of construction and improvement and the cost of maintenance. The cost of transport proper covers the use, management and care of the vehicles and the expenses of traction.

In the railway freight rates quoted by the writer both these elements are included, while the water-route rates include the cost of transportation proper only. To make the comparison a fair one, the freight rate for the water route should include the toll for the use of way. I have not the data at hand to determine what this toll amounts to for the water route from Chicago to New York. M. Colson states that for the navigable ways of France the average cost of transport proper should be doubled to allow for this element.* If we apply this rule to the water rates quoted by the writer, what becomes of the economy of the water transportation ?

The writer's views regarding the economy of inland water-transportation are, however, based, not only upon the statistics of our own country, but also upon the experience of other nations. He says :

"The water-ways of France and Germany also show economy of transportation when heavy, bulky materials are to be carried. The more the question is studied, the more desirable the use of water-ways becomes. It is evident by the great attention which many nations are bestowing on their navigable highways that there is something real, something tangible, in the economic results to be obtained by water-carriage."

Doubtless there is in many cases something real and tangible to be obtained by water-carriage, but it is certain that generally this tangible something is not economy of transportation. An analysis of experience in France seems to prove conclusively that between two given points the mean net cost of transportation is in general lower for

*Transports et Tarifs, par C. Colson, Ingénieur des Ponts et Chaussées, etc., page 224.

the railway than for the water-way. Moreover, it has been asserted by high authority that, owing to its advantages of speed, regularity and general convenience, transportation by rail is preferable to transportation by water, even when the cost of the former exceeds that of the latter by about 20 per cent.

If these views are correct, it is hardly necessary to adopt the author's theory to account for the temporary abandonment of artificial water-ways in this country. This theory is briefly as follows: When railways first came into existence the people were carried away by the marvelous results that were predicted and their feelings overpowered their judgment. The capabilities of the canals had not been fairly tested. Under the influence of this sentiment, the railway system was rapidly developed, the best ability of the country being enlisted in its service. In addition to this sentimental obstacle, the canals had to contend against the constant and vigorous attacks of the railway managements, which, although composed of the ablest men in the country, become "ignorant and prejudiced" when water transportation is in question. The canals were not improved or developed so as to utilize their advantages to the full extent; and, unable under these circumstances to compete with the railways, they were abandoned, or, if maintained, they steadily lost ground as carriers of freight.

This is not the way I read the history of American transportation. When the railway was introduced, the people, with their usual sagacity in business matters, saw that the new method of transportation presented advantages which artificial water routes did not possess. It was available at all seasons of the year; it saved time; it was adapted to both light and heavy freight and to passenger traffic; it furnished regularity of service; and, finally, it gave the lowest net cost of transportation. The people could not be fooled into the idea that they could reduce the price of transportation by taking part of it out of one pocket and part out of another. They developed and improved the railways at the expense of their own earnings and because they believed it would pay. They did not develop and improve the canals because the earnings of the latter would not bear the expenses of development and improvement, and because they thought it would not pay. They neglected or abandoned the canals because they rightly believed that under the existing conditions of the transportation of the country any other course would be a financial absurdity.

I am strongly impressed with the idea that, although the American people may lack the wings of theory, they somehow or other generally manage to arrive.

I cannot agree with the writer that the abandonment of the canals of Pennsylvania "partook of the treachery of Judas." For my part, I can see a canal abandoned and in decay with more equanimity than I can see it foolishly and uselessly operated at the expense of the people. Incidentally it may be remarked that nine of the Pennsylvania canals are still operated, although doubtless they cannot be considered financially successful if cost of construction is taken into account.

Does it follow from all this that the development and improvement of inland water transportation is inadvisable? By no means. There are many circumstances in which such a policy is of great benefit to the public interests. In some cases natural water-ways exist which may be made available and convenient for navigation at a cost which will give a net cost of transportation for heavy freight sufficiently low for competition with the railway lines. The improvement of such water-ways is generally wise on economical grounds. In some other cases, even where improvement involves a net cost of transportation greater than that obtained from the railways, it may be sound public policy to improve a water-way at the public expense. For example, such a route may be needed for a heavy traffic which could not otherwise be maintained and which may be of great importance to the manufacturing or other interests of the country. Such a route again may be desirable to prevent the diversion of heavy transportation to the sea to some existing water-route in a foreign country. Finally, it may perhaps be wise in some cases to maintain water-routes at the public expense to control freight rates and as a check upon railway combinations; but if this is the only object of construction, the question arises whether the Government would not be equally justified in constructing and maintaining a competing railway for this purpose.

It is important to observe that the desirability of improving the routes of water transportation in any country increases with the density of the population and the development of the railway system. In a country which is not densely populated the business of transportation can generally be handled by railways more economically and satisfactorily than by water-ways, unless indeed there are natural water-ways which can be utilized at small expense. There are, however,

natural limits to railway expansion; and when a country becomes so densely populated that its transportation business cannot be conveniently handled by railways, it may become a matter of high public policy to utilize the water-courses to their full extent. In France and Germany this condition of things seems to have been nearly reached. According to Colonel King,* France in 1885 had a population 13 times denser than that of the United States, with 1 mile of railway to 11 sq. miles of territory. The population of Germany was 15 times denser than the United States, with 1 mile of railway to 9 sq. miles of territory. It will be readily understood that the question of the comprehensive development of a system of water-ways stands upon a very different basis in these relatively small, compact and densely populated countries than in our own extensively and sparsely populated country. Even in these countries the wise limit for the extension of water transportation is still the subject of earnest consideration and debate; and in France, as is shown by the author himself, the improvement of water-ways was seriously neglected for a period of 25 or 30 years after the introduction of railways.

The author is in error in supposing that no systematic plan has ever been proposed for the development of the water-ways of the country. In fact such a plan was very fully considered by the Senate Committee on Commerce, and was the subject of very elaborate investigations and reports by the Corps of Engineers from 1874 to 1881. These reports now repose in their appropriate pigeon-holes, awaiting the time when the conditions of the country justify the serious consideration of such schemes; a time which, in my opinion, has not yet arrived. A special committee of the Senate for the consideration of this subject still exists.

In fact the method of procedure still followed in this country, to which the author so strongly objects—the method of considering each project for a river and harbor improvement upon its own merits, with special reference to existing needs and conditions, and with incidental reference only to the probable necessities of the distant future—is the only method which can be wisely followed for a long time to come. A comprehensive scheme of improvement at the present time is simply a paper scheme. It may be of interest to theorists as a hydraulic diversion, but it can be of no present value to practical men who are seeking results of immediate benefit to the country.

* River Transportation and the Muscle Shoals Canal.

Finally, I think the author unintentionally misrepresents the character and history of our system of conducting public works, and fails to recognize its very remarkable progress and results. Although considerable work of river and harbor improvement was executed at the expense and under the direction of the General Government previous to the Civil War, a system for such improvements can hardly be said to have existed. The present system may, I think, be considered to have originated about the year 1866. A system for the projection and execution of public works cannot be improvised, especially under a representative form of government, but must be slowly developed, in conformity with the character and needs of the country; and it can only be perfected by long experience. Accordingly, it is not a matter for surprise that at the commencement of the system and during the early years of its history it should present grave defects, and serious obstacles to its improvement should be encountered. Some of these obstacles and defects may be briefly noticed.

1. The right of the General Government under the Constitution to execute works of river and harbor improvement was very seriously questioned. Even admitting this right, there was great difficulty in establishing the limits of the powers and duties of the Government and those of the States. For example, the Constitution places under the control of the General Government certain matters in which the interests of the whole country are directly involved, leaving the management of purely local affairs under the control of the States. But there seems to be no direct provision for the control of matters in which two or more States are involved, to the exclusion of the rest of the country. If, for the purpose of illustration, we suppose it to be desirable to construct a comprehensive system of levees along the banks of the Mississippi for the purpose only of protecting the adjacent lands from overflow, and without reference to the needs of navigation, it would be difficult to find authority for the execution of the work by the General Government; and yet, in the nature of the case, it is a work which cannot be efficiently accomplished by the States interested.

As regards this question, it is important to notice that there is a marked distinction between the improvement of maritime ports and their approaches and the improvement of routes for inland transportation. The General Government derives a large revenue from the duties on foreign commerce, the collection of such duties by the States being

expressly prohibited. It therefore seems reasonable that the General Government should bear the expense and assume the duty of creating and maintaining facilities for communication with foreign countries. The improvement of inland navigation by the General Government to facilitate commerce between the States and to regulate rates of transportation by interfering with natural development under competition is an entirely different question, and one which is still worthy of serious investigation and thoughtful discussion.

2. Theoretically the improvement of a harbor or water-way promotes the general commerce of the country and is therefore a benefit to the whole nation. But our country extends over so great an area, and its commercial interests are so diversified, that practically each improvement is a much greater benefit to its immediate locality than to the rest of the country. The extent of the improvements desired being very great and the amount which can be obtained from the Treasury in any one appropriation being necessarily limited by financial and political considerations, it naturally follows that there is a lively competition between the congressional representatives of different parts of the country to obtain provision for improvements in their own localities. I know no way in which such competition can be prevented and I do not think it ought to be prevented if it could be. It seems to be the best if not the only way to insure due consideration for the interests of every part of the country. But in the early days this competition resulted in bargains between members of Congress by means of which unwise schemes of improvement, which doubtless involved a waste of the public money, were occasionally adopted. The extent of this evil has been very greatly overestimated, the amount of money unwisely expended under the orders of Congress being very much smaller than is generally supposed, even in the early years; but the evil did undoubtedly exist. At that time our system was altogether too "comprehensive!"

3. As a very large number of improvements were demanded, and as the total amount available for appropriation was limited, it became the custom to scale down these amounts allotted to the various works frequently below the limits of profitable expenditure. Thus the total cost of the improvements was often greatly increased.

4. At the commencement of these works of improvement after the termination of the Civil War, the number of Government engineers

who had much experience in conducting such works was comparatively small. The work of the Corps of Engineers during many preceding years had been principally of a military character, and the methods of administration and experience of the corps were better adapted to military than to civil engineering. The officers of the corps were carefully selected from the graduates of West Point just as they are at the present time, but no provision was made for training them to design and execute civil works, except the elementary course of instruction at the military academy. Under the circumstances it was inevitable that for awhile mistakes should occur, and it is not surprising that Congress did not place full confidence in the advice and opinions of inexperienced engineers. Undoubtedly the committees of Congress did in these early days occasionally formulate opinions on purely technical subjects which they were not professionally qualified to consider, and thus to some extent "usurp the functions of the executive," as the author charges.

If time and space permitted it would be interesting to trace the development and improvement of this system through the past quarter of a century; but I must confine myself to a brief description of the system as it now exists. The acts of Congress which provide the funds and regulate the execution of works of river and harbor improvement by the General Government are framed by two committees—the Senate Committee on Commerce and the House Committee on Rivers and Harbors. The Senate Committee consists of 13, and the House Committee of 15, members. They are carefully selected, so as to represent all the commercial interests of the country. The majorities of these committees are composed of men who have had long experience in the consideration and direction of legislation for public improvements. The Chairman of the Senate Committee has had a continuous experience of 22 years, and the average length of service of the members is, I believe, at least 12 years. The Chairman of the House Committee has had a continuous experience of 12 years, and another member has had 18 years of continuous service. Although there are more changes in the House than in the Senate Committee, it contains, I believe, only five or six new members, the average length of service of nine members being about eight years. These are the men with reference to whom the author asks the question: "How can legislators, who rarely retain their seats for more than one or two terms, act intelligently on such matters ?"

These committees are, to all intents and purposes, commissions to consider and recommend projects for works of public improvement. They exert an influence upon legislation which could not possibly be exerted by commissions constituted in any other way, and they maintain the closest connection with the people and the States, a matter of great importance under our form of government.

The information upon which these committees base their action is furnished by the annual reports of the Chief of Engineers and the accompanying reports of officers in direct charge of works, which are prepared with great care in accordance with a carefully devised system, and which include commercial statistics for each locality. In cases of importance, special boards or commissions are organized on which civilians are sometimes associated with the Government engineers. In other cases permanent commissions have been created for the continuous investigation and direction of important projects. When further information is required, special reports on specific points are called for, and the committees frequently pursue their investigations through personal conferences with the Chief of Engineers, the division engineers, the local engineers, and persons interested in the work under consideration. Individual members and subcommittees sometimes personally visit the works. Legislation is almost always finally determined by a conference of members from both committees.

The system now adopted to prevent unwise expenditure of the public money is as follows: The law provides for certain preliminary examinations to be made by the district engineer or some other engineer detailed for the purpose. This engineer and his superior, the division engineer, are required to report to the Chief of Engineers whether, in their opinion, the harbor or river under examination is worthy of improvement by the General Government, giving the facts and reasons on which their opinions are based, including the present and prospective demands of commerce. They are also required to submit estimates for the cost of survey, and the preparation of plans for improvement. The Chief of Engineers is required to submit these reports, with his own views, to the Secretary of War. The latter is required to submit them to Congress, where they are printed for the information of the committees and the other members.

After the consideration of these reports, if Congress so orders, the survey is made and the project is prepared; but no work can be done

until Congress has duly considered the survey and project, and has authorized the work and appropriated funds for its commencement. This method of procedure has resulted in the rejection of a great many doubtful schemes, many of which doubtless would have been undertaken in earlier years. In recent river and harbor bills the percentage of unwise appropriations (if there are any) is probably as small as can be obtained under any human system.

The evils resulting from inadequate appropriations have been met by the adoption of the system of so-called continuous contracts, whereby the making of contracts for complete improvements to the full extent of the estimates is authorized, the work to be paid for as appropriations may from time to time be made by law. This system is still an experiment, but it has already been adopted for about seventeen of our most important and expensive works.

As regards the formation of opinions on purely technical subjects, the committees are now almost wholly guided by the advice of Government officers or other professional engineers. Looking back to the state of our engineering knowledge and experience twenty-five years ago, the exercise by congressmen of independent judgment in technical questions does not seem to me surprising. The general confidence which the committees now repose in the officers of the Corps of Engineers rests upon the fact that for a quarter of a century we have had practical and generally successful experience in the conduct of public works; that our methods of administration have been perfected and adapted to the system; that our theoretical instruction has been increased by a three years' course of training at the Engineer School, in addition to the four years of preparatory education at West Point; and finally (and most important of all), that our opinions are believed to be unbiased and impartial, and are confined strictly to professional questions, except when we are directly required to report upon questions of public policy.

But would it be an improvement if these committees were partly composed of professional engineers? In my opinion, most emphatically, no. The members of the committees have all the ability and knowledge necessary to fully understand any project the engineers may lay before them. They do not pretend to be engineers, and therefore they rely on the Government engineers for the solution of technical problems; but in practical questions of finance, commerce and public

policy, they are, as a rule, vastly superior to the engineers. On the other hand, if a Government engineer should advance an uninvited opinion upon a question of public policy, he would not only violate a well-recognized principle of professional etiquette, but he would disobey positive orders. The established rule and custom of the Corps may be illustrated by the following quotation from the general order giving instructions for the preparation of the annual reports for this year:

"The laws do not call for opinions as to the propriety and expediency of making appropriations; but they do require facts, both as to the cost of works and as to the commerce interested, in order that Congress may determine the question whether the general commerce of the country will be promoted by the several works of improvement."

It is this clear division between the legislative work of the committees and the professional work of the engineers (a division which seems to be growing sharper year by year) which I consider the most admirable feature of our system.

Looking back over the past quarter of a century at the results attained, not only in the improvement of our executive and legislative methods, but also in works of improvement actually accomplished, I feel that the United States has no reason to be ashamed.

In conclusion, I desire to express my high opinion of the character and ability of the congressional committees in charge of this subject. For some years past I have been frequently associated with these committees and their individual members, and their patience and industry in investigation, and the general wisdom of their decisions, has won my respect and admiration. Commissions for directing public policy in river and harbor works, better adapted to the needs and conditions of our own country, and more thorough, efficient and intelligent in the execution of their work, could not be desired.

Edward P. North, M. Am. Soc. C. E.—Referring to Captain Mahan's very valuable paper, "Inland Transportation," I wish to call attention to the assertion, "Efforts are made to improve certain streams, but each stream is treated simply as a problem in hydraulics and not as a part of a problem in commerce or transportation," and to cite in that connection the case of the improvement of the River Main from Frankfort to the Rhine, about 22 miles.

The freightage on this river, which had a governing depth of less than 3 ft., fell, through the competition of the railroads, from 492 000 tons in 1840 to an average of less than 10 000 tons for the years 1880–

82, when it was decided to improve the stream. Although the governing depth in the Rhine is only 6 ft., the German engineers decided to have a depth of 8½ ft. on the miter-sills of the locks and completed the work as far as the construction of the dams, by-passes and locks in three and a half years, though the full prism excavation was left for a year or two, and the locks have since been lengthened from 246 to 1 165 ft.

The great increase of traffic to Frankfort is shown in the following tables taken from the "Year Book" of the Frankfort on the Main Chamber of Commerce for 1892, viz.:

Through the canalization of the Main and the harbor improvements, the freightage of the Main, exclusive of rafting, has increased as shown below:

Years.	Tons.	Ton-Kilometers.
1880–82	9 442	311 586
1887	494 193	15 352 452
1888	696 759	20 551 352
1889	939 446	29 159 283
1890	1 129 039	34 807 411
1891	996 919	30 239 351
1892	1 204 533	36 863 819

Attention is called in the "Year Book" to the fact that the ton-kilometers on the Main have increased from 1882 to 1892, 118 times. That this increase of river traffic has not been at the expense of the railroads, is shown by the subjoined table:

The freightage by railroad and water in Frankfort, without transit and raft traffic, was as follows:

	Water and Railroad. Tons.	By Water. Tons.	Percentage.	By Rail. Tons.	Percentage.
1884	1 014 519	150 514	14.8	864 005	85.2
1885	1 047 845	150 805	14.1	897 040	85.9
1886	1 088 047	155 957	14.3	932 090	85.7
Average, 3 years	1 050 137	152 425	14.4	897 712	85.6
1887	1 373 691	360 063	26.2	1 013 628	73.8
1888	1 748 733	516 798	29.6	1 231 935	70.4
1889	1 911 758	577 610	30.2	1 334 148	69.8
1890	2 103 171	697 351	33.1	1 405 820	66.9
1891	2 045 268	577 165	28.2	1 468 103	71.8
1892	2 211 601	709 118	32.0	1 502 483	68.0
Average, 3 years	2 120 013	661 211	31.2	1 458 802	68.8

It will be noticed that, comparing the two three-year periods, the increase of all freightage was nearly 102% ; that by water was 333.7%, and by rail, 62.5%, but the aggregate increase by rail was one and three-quarter times greater than that by water. This is possibly the most successful instance of river improvement on record, taking both the time in which it was made and results into consideration.

In contrast to this we might instance the Tennessee River, which in the main stream and its tributaries presents a length of about 1 000 miles that under improvement would be available for navigation. The Muscle Shoals, 255 miles from the Ohio, have been under improvement since the General Government gave the States interested a land grant, in 1827, by the aid of which a canal of insufficient length, but with 6 ft. on its miter-sills, was completed, but afterwards abandoned. Since 1871 the improvement of this river has been under the direction of the General Government, and according to an article in the *Railroad Gazette* of November 14th, 1890, $3 112 500 had been appropriated up to that time, "and the interest account compounded semi-annually, at 5% per annum, amounts to $2 145 300, or about two-thirds of the cost." At this time I believe the canal had not been used commercially. This computation of interest was probably made on the supposition that the expenditures had been uniformly distributed. In the case of the Main, such interest account, on the same supposition, would have been only about 10% of the cost.

There is also another noticeable difference between the practice on the Main and that on the Tennessee. The depth on the miter-sills of the locks on the Main is 37.5% greater than the governing depth in the Rhine, to which the Main is tributary. This allows navigation to Frankfort by large boats that can be used only in high water, and is a strong incentive to further improvement of the Rhine itself. On the Tennessee, however, the depth on the miter-sills is for the present canal only 5 ft., instead of 13, or more, as would apparently have been adopted if the same views as to the value of large and powerful boats had governed the engineers of both countries. As the navigation past the Muscle Shoals is in part through a trunk, by-passes are not available, so that the through traffic is restricted to boats of 5-ft. draft. Such boats lack the capacity necessary for cheap freights, nor are they powerful enough to tow barges in high water.

Considering the length of the Tennessee and its affluents, with the

potential wealth of the valley drained, this does not seem a valuable issue of a study in hydraulics, and as a part of a river system over 15 000 miles in length it must be of less value to the transportation interests than to those rigorous economists who hold that it is cheaper to burn the steamboats on the Mississippi and its affluents than to expend money in river improvements.

There seems no other valid reason for expending money in river improvements, except a fair commercial expectation of increasing transportation facilities. With our present machinery for production there are commodities enough in the world, and the true benefactor is he who reduces the cost in time and money of assembling the raw products of manufacture and their distribution to consumers. For, under all circumstances, gains made by, or through, transportation are as inimical to the interests of a country in which they are made as gains made through reductions in the pay of laborers; as the cost of transportation must be taken in some ratio from the resources of both producer and consumer. Our wonderful record as producers and consumers—a record which shows that with 65 000 000 out of 1 500 000 000 inhabitants of this world, the people of the United States produce and consume over one-third of the iron, cotton and copper, and consume about a quarter of the wool and sugar produced – is greatly due to the fact that we have been reducing the cost of transportation per unit as no other people have, and at the same time bringing the producer and consumer nearer together, and that they both have secured larger returns for their expenditures of labor and money.

It is remarkable that as the producer and consumer have been brought nearer together, both the volume of freight and the ton mileage has increased. At the same time, the net earnings of our railroads have not been materially decreased. In fact, in some instances, they have been increased. An investigation made, preparatory to presenting a paper to the Fifth International Congress on Internal Navigation, "Notes on the Relations between Railroads and Water-Ways in the United States," showed that in the case of seven trunk lines, viz.: the Boston and Albany, the New York Central, the Erie, the Pennsylvania, the Pittsburgh and Fort Wayne, the Lake Shore and the Michigan Central railroads, the average freight rate per ton-mile fell from 2 cents and 9 mills in 1865 to 6.8 mills in 1890, and that the average net incomes of these seven trunk roads increased from 20.207% in 1865

to 31.95% in 1890, with less than one-quarter of the receipts for like services in 1890 than in 1865. And the average values of the stocks of these seven companies was virtually the same at the two periods, although their stock had been increased by 236%, and the bonds by 193%, in the interim.

It will be immediately seen that these roads are parallel to, or in continuation of, the freight rate through the Great Lakes. This reduction of freight rates to 24% of what they had been 25 years before, with the consequent national prosperity, was more due to the competition of the water route than to any other one influence, if not than to all others. The above-mentioned railroads were competing with the most capacious water route of the country. The great reductions in railroad freight rates were almost invariably inaugurated on these lines, and have passed to all the roads of the country; these trunk lines, at least, have increased their net earnings. In this instance, to borrow Captain Mahan's apt quotation, we have scattered and increased our gear through the great industries rendered possible by the low freights. An insufficient improvement of a water-course may, however, attack the money invested in a parallel railroad by reducing freight rates just enough to diminish net earnings without reducing the charges enough to develop greater production. This is the case of the man that withholdeth more than is meet, but tendeth to poverty.

Our system of internal navigation, on the whole, however, in spite of the development on a few of the main lines, can, as Captain Mahan observes, scarcely be called satisfactory. The minor streams are unimproved; the 15 000 miles of water-courses in the Mississippi Valley are unconnected with the nearly 3 000 miles of other rivers emptying into the Gulf on the one hand, and inadequately connected with the waters of the Great Lakes on the other. The production of the Saskatchewan Valley should be made tributary to the prosperity of Duluth, through the Red River of the North, and a competent canal. At the same time, both the lake and river systems of the Interior should be tied together and with the waters running into the Atlantic by waterways fitted to pass large traffics. These works would distribute the advantages of cheap freights over nearly all of our country to the great increase of employment and values.

Our experience with the Inter-State Commerce Commission shows the futility of trying to reduce freight rates by prohibitive legislation,

or at the command of a select body of men who are supposed to possess special fitness for the purpose. It might be violent to say that this commission, as soon as it was fully organized and in working order, succeeded in arresting the decline in freight rates which had been virtually continuous for 23 years. But, as a matter of fact, this decline was then substantially stopped. And apparently we have no insurance against following the example of Great Britain in freight rates—viz., no amelioration in the charges for 38 years—except through a radical improvement of our water-ways.

If this contention is even partially accepted it must be admitted that a careful study of the problem should be made by a commission fully competent, both by technical and commercial knowledge, to deal with a question that has so much influence on our prosperity. In the discussion following the reading of Mr. Corthell's paper, "The South Pass Jetties—Ten Years' Practical Teachings in River and Harbor Hydraulics," I recommended (*Transactions*, p. 243, Vol. XV) a report by a mixed commission, consisting of, say, three civil and three military engineers, with four persons interested in the commerce and industries of the locality, as precedent to an appropriation for the improvement of a stream or harbor, and that Congress should make the appropriation necessary for the completion of the work, so that it could be conducted with the speed necessary for economy. One part of this recommendation, viz., as to appropriations, has in some instances been adopted by Congress.

The very full and lucid paper contributed to the Engineering Congress of the Columbian Exposition, 1893, by M. l'Inspecteur-Général des Ponts et Chaussées Guillain,* recounts an error that is probably irreparable in planning the improvement of the French rivers and canals. Yet every question was apparently fully considered, and a judicious decision arrived at as to all except one. A scientific system of routes has been selected, connecting the different river valleys and also forming great through routes for transportation; the size of the locks has been (except on the Seine between Montereau and Rouen) made uniform, so that when the improvements are finished a boat loaded on any of the canals of France can reach the frontiers and most of the seaports. And this has resulted, on the canals improved, in a gain in kilometric tonnage of 109% between 1876

* *Transactions*, Vol, XXIX, page 1.

and 1891. And M. Delaunay-Belleville, in his paper presented at the Fifth International Congress on Inland Navigation, says that the augmentation in kilometric tonnage has been 48% for navigation, as compared with 9.3% by railroad, for the ten years ending with 1890.

The error in judgment, and apparently the sole error, made in planning this system of improvements was that the size of the boats is rigorously limited to 300 tons, except as to the Seine below Montereau, where boats of 1 000 tons may pass. These gains in water-borne traffic were made against the highest freight rates collected in any manufacturing country except Great Britain, and most of the canal traffic of France would be transferred to the railroads if their freight rates were no higher than those charged in this country. This is a very serious matter for France, for, if she reduces the freight rates on her railroads when they fall into the hands of the Government, she will impair or destroy the money invested in the canals.

It is a doubly serious matter for France, as the standard size of locks on the improved German water-ways pass boats of 800 tons. So that, unless France gives her manufacturers low railroad freights, which will destroy her investment in inadequate water-ways, she will be still further left in the race, through the cheaper internal freights of Germany. "It is possible to raise money in Europe for new guns, but not for new locks." Blame for the adoption of 300-ton boats on the French canals has not, to my knowledge, been located; but it does not seem possible that boats of that size for a maximum could have been selected if forwarders by water and presidents of Chambers of Commerce had been freely consulted, and their advice had carried due weight.

It is unpleasant to differ from Captain Mahan in his estimate as to the values of legislative and executive judgment. But whether the present capacity of French water-ways is the result of executive and administrative judgment coercing the legislative or not, he will remember that of the five vetoes of River and Harbor bills by various chief executives, four were strictly political vetoes, while the last and only one since 1854 was not in any sense a partisan proceeding, but was by one who thought he believed what the agents of railroads and bankers assured him he did believe. The veto was against the traditions of the party to which the vetoer belonged, and was rewarded by the claque of newspapers which has so long and vigorously opposed freight

reductions, and so far as he was able by this veto he stopped the reductions in freight rates then in progress. Our legislators have made no such mistake.

Nor is the Corps of United States Engineers entirely without members who, to adopt another of Captain Mahan's quotations, "think they think what they are taught," and insist on making improvements which do not improve, and fail to exceed in gravity the mistake of the French only through the fact that their mistakes are made in a country rich enough to buy both new guns and new locks. Possibly they are too much influenced by the fear that some man or locality may make money through the proposed improvement, a fear that seems easily implanted in breasts of a certain order by those in the present enjoyment of profits. A blending of such conservatism with the business element of the locality sought to be served by the appropriation could hardly fail to be beneficial to the interests of the country.

It is suggested, in conclusion, that public opinion is more easily enlightened by object lessons than by arguments. Disseminating knowledge of the fact that freight carried on the lakes during the census year cost less for transportation by $135 000 000 than an equal ton-mileage by railroad, and that that sum is equivalent to a dividend of 66% on the total expenditures by the general Government for the improvement of all rivers and harbors, has countervailed a great deal of newspaper mendacity. As a result, General Poe found enthusiastic support to his request for immediate money enough to deepen the Lake channels to 20 ft., but it is doubtful if an appropriation for 17 ft. in depth would have passed so quickly. If the administrative officers of our Government will propose improvements which by their capacity and convenience will cheapen transportation over large areas and develop increased production, they will call attention to the beneficence flowing from the recommendations of broad-minded men, those who really influence the great powers of Nature to the service of men; but a recommendation to increase the depth of a 10-ft. river to 11 ft. will develop no enthusiasm.

Captain F. A. Mahan.—Major Raymond seems to assume that I have confused the freight rate with the net cost of transportation, because I omit to make a specific distinction between the cost of carrying and the cost of construction, improvement and maintenance

of the way. Perhaps I took too much for granted as to familiarity with the figures of the Erie Canal which serves as my gauge in speaking of canals.

Up to the year 1882, when tolls were done away with on the Erie Canal, the receipts of various sorts amounted to $127 815 823 47. The cost of construction and maintenance was, up to the same year, $80 341 767 87, leaving a balance of $38 374 055 60. Improvements are evidently included under the cost of construction, to judge by the figures under this head. The items of construction and maintenance during the six years from 1886 to 1891, inclusive, amount to $6 812 731 50, an average of $1 135 455, or about 3% of the excess of the receipts over expenditures. Hence the investment of the above-mentioned excess at 3% would have given the funds necessary to carry on all the work which has been done during the past few years. This use of the earnings of the canal is a perfectly legitimate business transaction, and on this basis the tolls could be done away with and the canal be made free. In other words, the State pays the tolls for the use of the way out of past profits.

I do not agree with Major Raymond when he says "they (the people) did not develop and improve the canals because the earnings of the canals would not bear the expenses of development and improvement, and because they thought it would not pay." So far as I am aware, the canals of the State of New York are about the only ones which have been kept up at all, and on the Erie Canal alone can the canal experiment be said to have been fairly tried.

It may be objected that I make no allowance for stockholders' profits, if this term may be used. This is found in the increased valuation of property, in the growth of cities, in the general development of the interests of the State and its people. The Erie Canal must be deemed a success, and a marked success, from all these points of view.

As for the saving of time by rail I must say that it is not so great as many people suppose. I remember that when I was in charge of the work at Buffalo it took as long to send cement by rail from New York to Buffalo as it did to send it by canal, unless it were sent by fast freight and extra price paid therefor.

Whether the railway gives the lowest net cost of transportation is a question which is not at all settled in my mind on certain classes of freight. Railway transportation cannot compete with transportation

on the ocean, the great lakes, the sounds, or the large rivers. That, I believe, is universally admitted. A stream nearly dry during one-half of the year, and frozen over during the other half, could not compete with the railway. Between these two extremes there is a water-way which will carry as economically as the railways, and even more so. The question then is, can an existing canal be improved so as to reach this condition ? Some can, I trow ; some cannot.

Major Raymond mentions my speaking of the neglect of the improvement of the French water-ways for a period of 25 or 30 years after the introduction of railways, but he refrains from mentioning the extraordinary development of these water-ways since the limitations and uses of railways are more fully understood.

If I have misrepresented in any way "the character and history of our system of conducting public works," it is certainly unintentional. I fully recognize the immense progress and the remarkable results obtained by the corps of which Major Raymond and myself have the honor to be members. And yet I cannot but believe that the work of the corps would have been far more successful had more attention been paid to its advice and recommendations, or if the corps had been called on to prepare extensive projects into which technical details enter largely.

I shall not attempt to follow Major Raymond further, as he has wandered rather afield from my subject, in order to correct, apparently, a false impression which he seems to fear I have caused. For this I thank him. But I will not yield to him in doing honor to the corps to which we belong.

Mr. North's discussion of my paper seems to reinforce my arguments. At the same time he differs with me on some points, *e. g.*, the value of legislative and executive judgment. When I spoke of the executive I meant not the President, but the offices of the executive departments. I maintain now that the executive officers of the Government have far better judgment of technical problems and details than Congress possibly can have, for the simple reason that they are constantly handling such matters and are giving daily and increasing thought thereto. For example, I hold that the Chief of Engineers and his officers are far better able to decide how the improvement of a harbor or of a basin of a river should be carried on than any congressional committee can be, and *a fortiori* than Congress as a body. Con-

gress can best decide whether such and such works may be done as matters of public policy, but the details can best be handled by the executive.

"That Congress should make the appropriation necessary for the completion of the work, so that it could be conducted with the speed necessary for economy" is a remark which has been repeated over and over again in the reports of the Chief of Engineers. For more than twenty years have I seen attention called by one officer or another to the inefficient methods of making appropriations, but apparently without result. To be sure, appropriations of a smaller amount are now made sometimes, and authority is given to contract for considerably greater amounts, but there is no promise or guarantee, expressed or implied, that the funds will be forthcoming when needed to meet the Government's obligations.

It happens, unfortunately, that our executive or administrative officers may not propose improvements of the sort mentioned by Mr. North at the end of his paper.

The River and Harbor bill, approved July 13th, 1892, provides "That no preliminary examination, survey, project or estimate for new works other than those designated in this act shall be made." This proviso effectually bars the way against "the administrative officers of our Government proposing improvements, which by their capacity and convenience will cheapen transportation over large areas and develop increased production," until they are invited by Congress to present such a proposition. When a report covering this ground shall be desired by Congress, there will be no lack of ability to prepare it on a thoroughly comprehensive scale. Our country is large. A comprehensive scheme of the sort mentioned must cover a large area. It will be vastly expensive and it will require many years to carry it into effect. In fact its cost will be so great as probably to insure its rejection before time shall have been taken to consider that a large sum divided by many years means comparatively small annual expense. Congress moves slowly, as a rule, but the past season shows that it can take prompt action when public opinion makes itself sufficiently felt. The first step, then, is to awaken the public to a realization of its position, the rest will follow.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

657.

(Vol. XXX.—December, 1893.)

SUBAQUEOUS ROCK EXCAVATION.—DISCUSSION ON PAPER No. 606.*

By O. M. POE and JAMES DEAS.

O. M. POE, M. Am. Soc. C. E.†—The description by Mr. Deas of the method adopted by Colonel Beaumont for subaqueous rock excavation on the Clyde in 1880 (*Transactions* American Society of Civil Engineers, Vol. XXIX, page 138) is interesting, and the information is valuable. A large amount of that kind of work has been done under my direction during the last ten years, and I am an ardent advocate of subaqueous blasting and the use of dredges for removing the broken rock, in preference to the old method by means of a coffer-dam.

In August, 1883, I was assigned to the charge of the work of improving Detroit River, at the Lime Kiln Crossing, and of the Hay Lake Channel, in St. Marys River, Michigan. I found subaqueous blasting in progress upon both works. At the former, the contractor, Mr. C. F. Dunbar, of Buffalo, N. Y., had in use an efficient plant of his own. At the latter, the plant belonged to the United States, but it was

* "History of the Conversion of the River Clyde into a Navigable Water-Way, and of the Progress of Glasgow Harbor from its Commencement to the Present Day." By James Deas, C. E., Engineer Clyde Navigation. *Transactions*, Vol. XXIX, page 128.

† Colonel, Corps of Engrs., Bvt. Br.-Gen., U. S. A.

much less efficient that Mr. Dunbar's, although the idea was borrowed from his.

The history of the adoption of this method of subaqueous blasting by Mr. Dunbar, may be briefly given :

In 1866 or 1867, Mr. Dunbar had undertaken the removal of rock from the channel at the harbor of Erie, Pa., on Lake Erie. He drilled and blasted a portion of the rock through the ice, in the winter time, and, when there was no ice, used a raft for the purpose. All the drilling was done by hand. The average depth of the excavation in the rock was about 6 ft.

In the fall of 1872, Mr. Dunbar, as subcontractor to Mr. George Neilson, of Belleville, Ont., a contractor under the Canadian Government, undertook the excavation of subaqueous rock at the harbor of Port Colborne, Lake Erie. An attempt to drill the rock by hand failed, because the rock was found to be so hard that three men could only drill about 1 ft. in a day, and the use of some more economical method was imperative. During the winter of 1872–73, Mr. Dunbar constructed a steam drilling machine, using the hull of an old dredge to carry the machinery. He fitted the scow with a vertical spud at each corner to "pin" it up and hold it firmly in position, and laid a track on the edge of the deck along which to move the drills. On this track were mounted two 5-in. steam cylinder percussion drills, arranged to slide up and down in a vertical frame overhanging the side of the scow. The drills were raised or lowered by means of a windlass operated by hand, and were moved along the track by means of crowbars. The scow was 50 ft. long, and by starting one drill at the end, and the other at the middle of the track, each drilling five holes, at intervals of 5 ft. he was enabled to drill ten holes at one position of the scow. Holes 9 ft. deep could be drilled without changing the drill bits for longer ones. As soon as a hole was drilled to the requisite depth, it was charged and blasted without moving the scow. When one row of holes had thus been disposed of, the scow was moved back from the face of the excavation a distance of 6 ft., and the operation was repeated. The depth of channel made was 14 ft., which required the removal of the rock to an average depth of 2 ft., the maximum depth being about 6 ft. The rock was a hard, black, flinty limestone which could not be broken by charges of ordinary blasting powder, but yielded to more powerful explosives.

The work proceeded successfully from June to October, 1873, when the work was stopped. In 1875 the Canadian Government re-let the work, but required a channel depth of 17 ft. The contract was awarded to Mr. Dunbar, who used the machine described above, and successfully completed the undertaking. The difficulties encountered were : 1st, the extreme hardness of the rock (Mr. Dunbar considers it the hardest he has ever excavated), and 2d, exposure to heavy seas, which required the use of a machine that could be readily and quickly moved to a place of shelter.

During the progress of work under this contract Mr. Dunbar devised his apparatus for charging the drill holes. It is equally ingenious and simple, consisting of a copper cylinder having a smaller diameter than the drill hole, and length sufficient to admit the entire cartridge. At its upper end it is attached to a piece of iron gas pipe long enough to extend well above the surface of the water. The copper cylinder is slotted on one side throughout its whole length, to permit the insertion of the cartridge with its exploding wires fastened to the upper end. When the cartridge, with its wires, has been passed into the cylinder, the latter is inserted in the drill hole, and a long pole is passed down through the gas pipe forming the extension of the cylinder. The whole pipe is then withdrawn, the pole being used to prevent the cartridge from coming with the cylinder. As soon as the charging apparatus has been entirely removed, the cartridge is exploded.

So far as I am informed, this was the first steam drilling plant used in this manner for subaqueous work. It was afterwards greatly improved in some of its details, but the essential portions have not been changed. Among the most important improvements have been the introduction (while engaged upon the Lime Kiln Crossing work in 1879) of a hydraulic ram, to move the drills, both vertically and horizontally, and the use of a steam pump to force water into the drill holes for the purpose of cleaning them of all débris.

The latter device was first applied at Middle Neebish, Hay Lake Channel, where the excavation was in a very coarse, hard sandstone rock. The continuous use of the water-jet during the operations of drilling obviated the difficulty resulting from the packing of sand (drill cuttings) around the drill bit, which was often sufficient to render it impossible to withdraw the bit from the drill hole. It also left the finished hole entirely free from sand or other material which

would otherwise prevent the cartridge from going to the bottom of the hole. Simple as this device is, it made the difference between complete success and partial failure of the method at this locality.

While Mr. Dunbar was employed upon the contract of 1875, Mr. W. G. Thompson was resident engineer of the Welland Canal, including the harbor of Port Colborne, and Mr. C. James was the assistant engineer in charge. If these gentlemen are still living, they will doubtless remember all the facts concerning the use of Mr. Dunbar's method for subaqueous blasting at Port Colborne Harbor.

Apparently Colonel Beaumont's adoption of a like method upon the Clyde in 1880 was entirely independent of any knowledge of what had already been done on Lake Erie in 1873, and again in 1875, as well as at Lime Kiln Crossing in 1879. If this be the fact, it must still be admitted that the credit of priority in the use of the method is due to Mr. C. F. Dunbar.

It may not be amiss to mention the prices paid for subaqueous rock excavation in recent years in this locality.

The last contract at Lime Kiln Crossing was taken by Dunbar & Sullivan, at $4.43 per cubic yard, measured in the bank. The material is a compact buff limestone. The excavation was made in a current averaging about 3 miles per hour, but frequently running with much greater velocity. The channel was narrow (only 300 ft. wide when the contract was signed), and used by a commerce averaging fully one hundred vessels per day, many of large size, which greatly interfered with the progress of the work and constantly endangered the contractor's plant, a fact which had its influence upon the price.

The last contract at Middle Neebish, Hay Lake Channel, was taken by John Hickler & Son, at $1.20 per cubic yard, measured in the bank, a condition being that the major portion of the material should be formed into a dike rising 4 ft. above the surface of the water. John Hickler & Son employed Dunbar & Sullivan to do the work, and, it is supposed, at a price sufficiently below that named in the contract to leave a margin of profit to the original contractors.

Dunbar & Sullivan are now at work upon two contracts for deepening the prism of St. Marys Falls Canal, one at $1.53, and the other at $1.47 per cubic yard, measured *in situ*. The material is Potsdam sandstone in strata of varying thickness. The depth of channel is 25 ft., and the average depth of cutting something in excess of 8 ft. The

canal is the highway for a commerce which last year amounted to more than 11 214 000 tons in 233 days. This passing commerce seriously impedes the work, but not to greater extent than was anticipated.

John Hickler has undertaken to excavate a channel 21 ft. deep, at Sailors Encampment, St. Marys River, through blue limestone containing a considerable portion of flint, the contract price being $2.43 per cubic yard, measured *in situ*, the average depth of cutting being something more than 6 ft. Subaqueous drilling and blasting have been in progress during the past season.

The prices noted indicate the economy of the method under discussion. It should be stated, however, that a coffer-dam at Lime Kiln Crossing would not have been practicable without such an interference with navigation as to be inadmissible, and at St. Marys Falls Canal the use of a dam would have been confined to the season of closed navigation, say, from December 1st to April 15th. Each winter's operations would have involved the construction of a new dam.

James Deas, C. E.—I have read Colonel Poe's remarks with much interest, and they are so full and explicit that they leave me nothing to say except that I was quite unaware until now, and I believe Colonel Beaumont was also ignorant of the fact at the time he undertook our work, that the mode of boring adopted by him at Elderslie Rock had been previously in use anywhere else.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

658.

(Vol. XXX.—December, 1893.)

IMPROVEMENT OF RIVERS.—DISCUSSION ON PAPER No. 607.*

By TH. REHBOCK, C. O. GLEIM, WM. MURRAY BLACK and W. R. HUTTON.

Mr. TH. REHBOCK (translation).—Mr. Franzius asked me to give a short abstract of his paper. I only regret that I have not enough knowledge of English to speak in that language. I will limit myself to give in a few words a general geographical description of the river, of the purpose, means and results obtained, up to the present moment, by the executed works.

The River Weser is considered abroad as a large river with considerable discharge, while, in fact, its lower maritime part only is of any consideration, so far as the volume of water is concerned.

The River Weser is formed by the union of the rivers Fulda and Werra at Münden, 227 miles above Bremen. This part of the river, called the Upper Weser, has a drainage area of 15 400 sq. miles, and an average descent from 1 : 2 000 to 1 : 6 000. The Upper Weser, although navigable over its whole length, is up to this date of no importance for traffic, because there is no navigable connection with the

* "Description of the Lower Weser and its Improvement." By L. Franzius, Ober-Bau-Director, at Bremen, Germany. *Transactions*, Vol. XXIX, page 173.

other German water-ways, and its depth is insufficient during dry seasons. Nevertheless, it is to be expected that at no very distant future, in connection with the execution of the Rhine, Weser and Elbe Canal, the Weser, from Bremen to Münden, will be transformed by canalization into a first-class inland water-way, which will connect the flourishing trade of Bremen with the Rhine and the Elbe.

The tidal course of the river begins at Bremen, and bears between this town and Bremerhaven the name of Lower Weser. The improvement of this part of the river is described in the paper before you. The Lower Weser has a length of about 40 miles. It receives from the Upper Weser a quantity of water, varying from 2 500 to 140 000 cu. ft. (70 to 4 000 cu. m.) per second, while the lowest part near Bremerhaven is subjected to a mean tidal rise of 10 ft. 4 in. (3.30 m.), corresponding to a volume of water of 230 000 cu. ft. (6 400 cu. m.) per second.

In former centuries sea-going vessels of small draught could navigate the Lower Weser up to Bremen, which town by this fact became one of the richest commercial places in Germany. In the beginning of this century, however, the bed of the river was in such a bad condition that no sea-going vessel could reach Bremen, and their cargoes had to be discharged into lighters at Vegesack or Brake.

While Bremerhaven, founded in 1830, with a sheltered roadstead and large dock-basins, satisfied the requirements of the transatlantic trade, the traffic to the town of Bremen was hindered so much by the bad state of navigation on the Weser, that in 1887 the State of Bremen decided to spend 30 000 000 marks ($7 000 000) in carrying out the project for the improvement of the Lower Weser, presented by Mr. Franzius, in order to gain a good access from the open sea for the new harbor of Bremen, constructed from 1885–1887, at a cost of about an equal amount.

The programme of the improvement of the Lower Weser is the creation of a uniform channel of 6 m. (nearly 20 ft.) in depth in the entire upper tide basin above Bremerhaven, and the maintenance of the conditions thus created by the power of the increased and regulated current, with little other assistance.

The means to accomplish this object is the formation of a regular river-bed, gradually narrowing up stream, guided by training-walls, which enclose the low-water section, and the execution of large dredging operations in order to attain the necessary depth.

At the beginning of this year more than 17 miles of training-walls and of closing walls were finished, and 600 000 000 cu. ft. (17 000 000 cu. m.) of soil had been dredged. The expenses up to this time amount to 24 000 000 marks (nearly $6 000 000). The propagation of the tidal wave is already so much facilitated, that the tidal range has been increased 2 ft. (60 cm.) at the Great Weser bridge at Bremen, and the flow of the tide lasts one hour longer than formerly.

The depth of water in the channel up to Bremen, which in 1886 was only 9 ft. (2.75 m.), is in this summer more than 16½ ft. (5.0 m.) at ordinary tide, enabling vessels like the North German Lloyd steamer Hanover, with 2 571 register tons, to come up to the harbor of Bremen.

The part of the estuary of the Weser from Bremerhaven to the open sea, 35 miles long, called the Outer Weser, has no well-defined banks, and bears the character of a gulf, being from 1.2 to 12 miles in width.

The depth of this part is at high-water time sufficient for the largest vessels, while at low water the navigation is hindered by a bar, five miles from Bremerhaven.

In order to remove this bar, which is the result of a split in the river, and to establish a fixed channel, the governments of Prussia, Oldenburg and Bremen adopted a project of Mr. Franzius and appropriated 3 000 000 marks ($700 000) for its execution. In 1892 a training-wall about 4 miles in length was built, reducing the width of the river more than half a mile, with the result of increasing the depth of the river more than 8 ft. (2.50 m.).

The project, the execution and the results attained in the improvement of the Lower Weser will be published, with all technical details, in the course of the next year.

C. O. Gleim, Cor. M. Am. Soc. C. E. (translating some remarks made in German by Mr. Rehbock).—Mr. Rehbock wishes to ask a question; that is, whether any apparatus for stirring the soil, in order to deepen the channel, has been employed in the United States? That is occupying their attention in Bremen at present, and he would like to get some information. Mr. Rehbock has just given the information in regard to the requirements on the lower part of the Weser. They have now a call for competitive plans—an international competition—for plans proposing an apparatus to stir the soil, so that, with the strong current they have with the tide, the sand would be washed out without dredging. He says no such apparatus has been used on the Continent, except at Dunkirk, which has not been successful.

WILLIAM M. BLACK, M. Am. Soc. C. E.—The only case I know of that kind has been the stirring up of the bottom by strong jets of water; this has been done with more or less success. In the case of the stirring up by dynamite, it was found that the material was started up very successfully, but the sand was deposited very soon and very close to the point from which it had been started.

Mr. REHBOCK (translated by Mr. Gleim).—The cases referred to by Captain Black are rivers where it is really an estuary near the sea. In the case referred to, on the Weser, they have a great deal of silt, and it is the silt that they wish to have stirred up and carried out, and not the sand. The bar is effected by the training wall that they have erected on the other side.

Captain BLACK.—There was a method of deepening bars in use on some of our western rivers some years ago by scraping. What was known as "the Long scraper" was made by fastening a semi-cylinder of heavy metal at the end of a handle, forming a tool in appearance much like the dipper of a dipper-dredge. This was attached to the stern of a scow and drawn across the bar. It was used successfully for a good many years, but the improvement made by it was found not to be permanent.

Another method used on the lower Mississippi now, just after the flood has passed and before the low-water channel has been formed, to cut through a temporary bar, is to drive piles directly along the middle line of the proposed channel, in a row, about 10 ft. apart. It is found that these induce sufficient scour to open the channel.

Another method has been tried on the ocean bars in the South. Charges of dynamite are sunk a few feet below the bottom, over the bar. These are exploded, and the sand thrown up by them is carried away, to a greater or less extent, by the tidal currents. The difficulty with all of these devices arises from the fact that the sand and heavy sediment are deposited within a short distance of the point from which they started.

On the Willamette River, bars have been cut through by propeller dredging. This is done by mooring a deep-draught steamer above the bar, with stern down stream. When the propellers are started ahead very rapidly, a channel is cut through very quickly by the strong currents produced. The same method of dredging I have seen used successfully in sand, where it was desired to obtain a deepening that was

* Capt., Corps of Engs., U. S. A.

only local. In all these cases, however, the sand is deposited a short distance from the place whence it started. I have seen that in hydraulic suction dredges. In using a dredge of this description in shoal water I have seen the sand deposited 10 ft. from the extremity of the outflow pipe. The deposit was made so locally and rapidly that it threw up a bank along the edge of the cut, showing how quickly the sand, raised up and moved in this manner, is deposited.

W. R. HUTTON, M. Am. Soc. C. E.—This paper is not a criticism of the work done upon the Weser. That is a successful work and therefore beyond criticism. It had for its object to make a navigable channel of a given width and depth from Bremen to the sea. The methods adopted procured a sufficient measure of success and, doubtless, at the smallest cost. This is true engineering; the proper and judicious application of the means to obtain a given result.

But to the writer it seems that some of the methods employed, judicious under the circumstances, have not the special merit attributed to them, and may lead the young engineer into trouble if unintelligently followed.

The method of improving tidal rivers was first analyzed and described by Mr. Caland, in preparation for the design of the Rotterdam water-way. He treats the entire river channel as an elongated estuary, arranged to cause the flood tide to ascend as far as possible from the sea and with the greatest possible range of tide, thus introducing the largest quantity of tidal water into the river-bed. For this purpose there is required a depth nearly uniform, and a width diminishing regularly from the mouth to the head of tide, in proportion to the diminished quantity of water passing through each succeeding section.

The paper under discussion appears to be a revision after another year's experience of one presented last year to the Paris Congress of Inland Navigation. The discussion upon that paper was confined almost entirely to the question whether the regulated channel should be formed and maintained by dredging or by scour, the Rotterdam water-way being cited in favor of the exclusive use of dredging as against the practice on the Weser, where a large quantity of material was moved by scour. The cases are not comparable. The most appropriate method was adopted at each place.

In the paper of last year it is recommended that all the secondary

channels or large arms of the river which are to be closed should be left open at their lower ends, that they may be filled by the flood tide, and thus contribute to augment the volume of water circulating through, the estuary, both at flood and ebb. This recommendation is not repeated in the paper of the present year, but is referred to here because it seems to have been approved by high authority in the Paris Congress, although not in conformity with the accepted Caland theory. That theory excludes all irregularities of width, everything which may diminish the volume of the ascending tide.

The cross-section of the New Weser consists of two parts, a deep central portion or low-water channel, in designing which Mr. Franzius has followed what we may call the Caland method, and shallow lateral parts, the bottoms of which are about at the level of low tide. The flood tide-wave passes up the deep central channel with a velocity proportioned to its depth.* The lateral beds being very shallow, their velocity will be insignificant, and during flood tide they will be "filled from the middle section and will empty themselves back into it during ebb tide, thus operating like elongated flushing basins" or lateral reservoirs.

This arrangement is exceedingly defective as a means of increasing the tidal power and range, and obtaining the highest results in the improvement of the river-beds.

The volume of the flood wave is diminished by the water abstracted to fill the lateral beds, its height and velocity are reduced and its *vis viva* lessened, to the injury of the whole river-bed above. Less water ascends to the head of the tide and less flows out with the ebb.

This feature, however, found no objection in the committee of the Paris Congress, and seems to have been in the mind of the engineer of the Seine when persistently defending the use of lateral reservoirs in the bed of the stream.

The effects of the side beds would be similar to that of the lateral reservoirs condemned by the committee, except that, being continuous, their consequences are not abrupt, but gradual, and therefore not perceptible.

---

* Green's formula referred to in the paper is $v = \sqrt{gt}$, t being the depth of the center of gravity of the section below water surface. The formula used by the engineers of the Seine is found to agree fairly well with observation, $v = \sqrt{g\,(H + h)} - U$, where v = velocity of tidal wave, H = depth of low water, h = height of wave, U = velocity of ebb current.

The real merit of the plan, as applied to the Weser, consists in the very large saving in cost, combined with a sufficient improvement of the navigable channel, rather than its efficiency in improving the navigability of the river.

Mr. Franzius has explained the method of determining the high-water and the low-water channels in other publications, not now accessible. In his judgment they should co-operate to increase the volume and the velocity of the water in the river.

The fact is not overlooked that the Clyde and other rivers of Great Britain and elsewhere have been improved to the highest degree by the use of low or half-tide training walls. In them, as a general rule, are present the conditions which justify the use of the low dyke, viz., the lateral beds or reservoirs are so near the mouth, and the mouth so large, that, 1st, the volume of water passing into the river above the reservoir is not diminished by the filling of the reservoir; 2d, the emptying of the reservoir into the sea does not obstruct the outflow of river water from above.*

* Amélioration de l'Escaut. Vème Congrès International de Navigation Intérieure.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

659.

(Vol. XXX.—December, 1893.)

HARBOR IMPROVEMENT.—DISCUSSION ON PAPER No. 608.*

By JOHN PURSER GRIFFITH.

Mr. JOHN PURSER GRIFFITH.†—There are in the British Islands many piers and breakwaters constructed on the same principles as those described in this paper on Leixões harbor, and I have been asked to give in a few words my experience of these structures. One of the most important examples with which I am acquainted is Holyhead breakwater. It is constructed in deep water and in an exposed situation. The masonry superstructure or backbone is founded on the rubble mound about low-water level, and is protected on the sea side by a great foreshore of large rubble stones, originally rising to high-water level. Twenty years of exposure to the storms of the Irish Sea has reduced the rubble foreshore to little more than a pebble beach, flattening down the foreshore and threatening the denudation of the rubble masonry foundations. The destructive action of

* "A Brief Account of the Building of Leixões Harbor." By Alfonso Joaquim Nogueira Soares, Inspecting Engineer in Charge. *Transactions*, Vol. XXIX, page 194.

† Of Dublin, Ireland.

the waves is increased by the vertical wall, the waves being increased in height and reflected on to the foreshore. The rubble is rolled about and transferred from one part of the breakwater to another, forming local accumulations, and sometimes entirely removed from that part of the work where it is most required. In the great piers which enclose the harbor of Dublin we have had some rather striking experiences of the effect of waves on rubble mounds. Poolbeg Lighthouse, the principal light of Dublin harbor, is at the extreme seaward end of the Great South Wall, which extends 3¼ miles into Dublin Bay. The base of the tower was formerly protected by a foreshore of rubble. Each winter's storms removed some of this, making it travel in the form of a stone wave landwards along the pier. Season after season this stone was brought back to the base of the lighthouse tower or replaced by fresh stone, the annual cost of so doing amounting to about £800. Some twenty-five years ago, concrete blocks, weighing 40 tons each, were built *in situ* on the foreshore in the expectation that as the foreshore was washed away the blocks would settle down and stop further denudation. Some of these blocks were actually carried away by the sea from the place in which they were built, and it was found necessary to gird the exposed side of the lighthouse with concrete blocks, each weighing 140 tons. These have succeeded admirably, and for years all expenditure in protecting the base of the lighthouse has practically ceased, and the blocks are now as perfect as the day they were laid.

On the north side of the harbor entrance there is a breakwater entirely composed of rubble stone. The outer end of this pier for several thousands of feet in length is a half-tide level wall, over which very heavy seas break, and yet the mound has practically remained stable for seventy years. These two examples seem to point out that a rubble mound surmounted by a vertical superstructure is in a condition of much less stable equilibrium than if the superstructure were omitted and the sea allowed to break over the pier.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

660.

(Vol. XXX.—December, 1893.)

IMPROVEMENT OF RIVERS.—DISCUSSION ON PAPER No. 609.*

By H. ENGELS, WM. MURRAY BLACK, W. R. KING and JOHN PURSER GRIFFITH.

Prof. H. ENGELS.—In presenting my paper to the Engineering Congress, I beg to add a few remarks to prevent a misunderstanding in regard to No. 1 of the conclusions contained in the paper. I do not wish to be understood as saying that rivers still undergoing the process of erosion are incapable of regulation. Such rivers I have declared adapted to canalization. But every canalization must be preceded by regulation. What I wished to emphasize is this, that in rivers of that class the attainable limits of navigability cannot be produced by regulation, nor permanent success gained by means of regulation. Improvements may certainly be effected by regulation in a river yet in the state of erosion; but, the river itself not having attained a permanent condition, the improvement of its condition effected by regulation

* "The Limits Attainable in Improving the Navigability of Rivers by Means of Regulation." By H Engels, Professor of Hydraulics, Royal Technical High School, Dresden, Germany. *Transactions*, Vol. XXIX, page 202.

cannot be expected to be permanent. For this reason it is impossible in rivers of this kind to determine in advance the limits of navigability attainable by regulation.

William Murray Black, M. Am. Soc. C. E.*—We may well believe from the evidence given by the structures on land that statical engineering is rapidly becoming an exact science. Where an engineer has to deal with the forces of Nature exerted through the winds, waves and currents, he is made to feel that as yet the accumulated knowledge of the world gives him very little exact data to guide him; the formulas are largely empirical, and this branch of engineering science is still in the experimental stage.

As emphasized by Mr. Engels, in all hydraulic work experience and common sense are most important factors in securing successful results. Each particular problem requires its own study, and an engineer cannot treat a river successfully without such study any more than a physician can cure an obscure disease without an intimate knowledge of the patient's characteristics.

The causes of the obstructions in a river's bed and the motion and fluctuations of the water in that bed must be determined carefully before it is possible to determine the extent to which the obstructions can be removed and the width and depth of channel which can be obtained and maintained. It is easily possible to destroy good existing navigable conditions in the upper reaches of a river by misdirected and ill-considered work below. The entire stream and its capacity for commercial uses should be considered together.

This paper is a timely and admirable discussion of this very important subject.

Mr. Engels notes the surface elevation of a stream along the concave shore of a bend. An example of this was given to me a short time ago by General Comstock, President of the Mississippi River Commission.

In a bend of the Mississippi River with a radius of one mile at a point where the river is 3 000 ft. wide, the centrifugal force may cause a difference of head of about 2 ins. on the two sides, higher on the concave. Along this concave bank the velocity is greater and the centrifugal force higher at the surface than near the bottom. The head of 2 ins. before mentioned then causes a downflow along the concave bank, pushing the lower strata of water out. Thus, a current is started

* Capt., Corps of Engrs. U. S. A.

toward the convex bank, carrying with it the eroded materials and piling them below the convex point.

Another phenomenon seen on the Mississippi River and perhaps observed on streams carrying a less volume of sediment is this. On the lower Mississippi, which carries a large volume of sediment, it is well known that a rise of surface is frequently followed by a rise of the level of the bottom, so that the increase of surface height by no means necessarily indicates an equal increase of depth. The thalweg of the Mississippi River at low water is a channel winding through the greatly broader thalweg of the stream at its high stages. The great increase of energy accompanying a rapid rise of the river is first expended in straightening the low-water thalweg. Flood water from the Ohio or Missouri rivers enters the Mississippi laden with sediment, and the total volume of sediment of the river is at once increased by the sediment eroded in straightening the low-water thalweg. As the water rises the low-water banks are overflowed, and the broader valley is filled. After the crest of the flood wave has passed, the current velocity is decreased and a portion of the sediment is deposited. The falling water then becomes more shoal in the wider thalweg of the high river, and the minor features of the bottom which, during the flood, had had but little effect on the surface and slope, begin to be noticeable. The velocity ceases to be uniform through long stretches. Erosion of the bed begins where the velocity is greatest, and as the channel there is deepened, this increase of velocity, accompanied by erosion, is transmitted up stream until the river has again excavated its low-water channel. I think this may be considered a fair statement of what takes place after a flood, and any one familiar with it knows that the channels of the Mississippi River are almost always changed after the flood and before the low-water channel becomes re-established.

I would very much like to have Colonel King give his experience on the Tennessee River, because that is a case not covered by Mr. Engels' paper, although it is a case which comes under the principles of that paper.

Colonel W. R. King.*—As I left that work some seven years ago, and have hardly thought of it since, my thoughts on the subject are a little scattered, and will not be as clear as they ought to be. It was only this morning that I received the first intimation that this paper would come up, or that I would be likely to have anything to say about it.

* Col., Corps of Engrs. U. S. A.

The Tennessee River is an exception, I might say, to all rules that are usually laid down for the improvement of rivers, and in getting the available depth the conditions are entirely different from any that are mentioned in Professor Engels' paper. The Tennessee is not a shifting stream; it is a "completed" stream, and the regimen is almost constant. In certain places there is a little washing, but generally the bed and banks, of rock and gravel, are not disturbed very much either by floods or by low water. To find the greatest depth that can be attained at reasonable expense, we proceed somewhat tentatively. At the shoal places the river has generally spread out, perhaps one mile wide, though its normal width is only about 1 000 ft., and we simply cut a channel of suitable width through the obstructions, at the same time narrowing the channel by wing dams, or by closing side channels, so as to keep the deepened channel filled to the required depth. If we should cut out the channel and make no contraction in the river, we would have a channel without water enough to fill it, and would lower the pool above so as to cause secondary obstructions to appear. We, therefore, attempt to contract the channel in width as much as we enlarge it in depth, and this can generally be done without applying formulas; we find in that way that we can improve the worst obstructions without producing obstructions elsewhere.

As to the greatest depth that can be gotten, we must have a certain width, and, knowing that and the low-water discharge of the river at that point, we can find out the depth of channel that we can effect.

Then we have another thing to look after—the rapid current. And to avoid this, the wing dams must be properly located, so as to back the water up to the obstruction without raising it much on the crest of the reef. If we undertake to get too great an increase in depth, the number of obstructions to be removed, as well as the cost of overcoming each separate obstruction, will increase very rapidly. In a river like the Tennessee, where the demands of navigation are limited, we find that if we attempt more than about 3 ft. it would involve too many obstructions. We have worked in that way, trying to improve the worst obstructions. The boats can take advantage of the higher stages of the water, and are not compelled by the necessities of commerce to use the extreme low stages. We have balanced the matter of cost and depth in that way, finding that it would not cost excessively to get, say, 3 ft., but that to get another foot would bring in too many secondary obstructions, and greatly increase the cost.

Again where, as at Muscle Shoals, we were obliged to resort to locks, we built the locks in such a way that boats of greater draft than can navigate the river at low water can pass them, because the locks can be used at high water; we found 5 ft. on the miter-sill was an economical depth, and would give ample accommodation to the commerce of the river. As the entire length of navigable water on the Tennessee and its tributaries is about 1 000 miles, the cost of radical improvement by lock and dams would be too great to be thought of, considering the extent of commerce involved.

Instead of being of a temporary nature, this work, as executed, is comparatively permanent, from the fact that we have a good foundation to build on. The dams are sometimes built of small stones. We found, if built up like ordinary walls, even if large stones were used, they would be torn out by one flood; but by laying the stones lapping on each other like shingles, they would remain.

The cost of this kind of improvement per mile is but a small fraction, say, 10%, of what is often spent on smaller streams in Europe.

There is a whole class of streams, like the Tennessee, where the conditions are entirely different from the conditions assumed in Mr. Engels' paper, and these remarks are mainly intended to prevent a hasty conclusion that because the Germans and French are unable to make certain permanent improvements on their rivers, we cannot do it with ours.

Mr. John Purser Griffith.*—When I first looked at Professor Engels' paper I must admit feeling rather like one of those described on page 221 of his paper, "who, on account of the title, have counted on an exposition equipped with great mathematical and speculative apparatus, who have expected that the question propounded would be solved in an abstract, theoretical manner." Such, Professor Engels predicts, "will be disappointed." I was disappointed, but very agreeably disappointed.

I found that the paper was not merely a discussion of the problem from a theoretical point of view, but was full of the results of the keen observation and sound judgment of a practical engineer. I have read the paper with great profit, and specially note the important distinction drawn between the "finished" and "unfinished" reaches of rivers and their importance where regulation works have to be con-

*Of Dublin, Ireland.

structed. The paper is full of matter worthy of close study, and in one particular seems to be unique, for I cannot recall having met with the influence of ground or spring water so fully discussed elsewhere.

Another point of great interest has been raised, namely, the use of models in solving certain problems connected with the flow of rivers and the transport of sand and silt. My friend the late Professor James Thomson, of Glasgow University, was, I believe, the first to investigate the problems connected with the flow of water round river bends by means of models. He also proved the existence of vertical upward and downward currents in rivers produced by the retarding influence of the bed of the stream, and showed that these currents accounted for the surface velocity not being the maximum velocity.

I simply refer to these experiments of Professor Thomson that Mr. Engels may be encouraged in his researches by the aid of models. Those interested in the subject will find Professor Thompson's experiments fully described by himself in the reports of the British Association.

Prof. H. Engels.—In reply to Colonel King, I have to say that my remarks—as I ought to have mentioned—refer to rivers with unstable beds. In the case of a river with a rocky, stable bed, and to this class, according to Colonel King, the Tennessee seems to belong, a depth can certainly be obtained depending only on the following elements: 1, available funds; 2, quantity of water at low stages; 3, admissible minimum width of channel; 4, admissible maximum rapidity of current; the last two conditions being governed by considerations of navigation.

As to the case of the Muscle Shoals, it does not seem to form part of the subject under discussion, being a case of canalization and not regulation.

Neither is the conclusion at which Colonel King arrives founded on the contents of my paper. Nowhere did I say that the Germans and French were unable to put their rivers in a good permanent condition by means of regulation. I only explained that this was impossible in the case of "unfinished" rivers; and this is equally impossible with American streams in case the latter are unfinished.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

661.

(Vol. XXX.—December, 1893

IMPROVEMENT OF HARBORS.—DISCUSSION ON PAPER No. 610.*

By JOHN PURSER GRIFFITH, WILLIAM MURRAY BLACK, WILLIAM P. CRAIGHILL and THOMAS W. SYMONS.

Mr. JOHN PURSER GRIFFITH.†—I wish to express the pleasure with which I have listened to Captain Black's paper. It opens up a subject of such importance to all nations that I think it finds a fitting place in this International Congress of Engineers, although, of course, our discussion must necessarily be of a very limited character. I think we shall all agree that in many respects the instances quoted by Captain Black of the improvement of harbors on the South Atlantic coast of the United States differ in many important features from the best-known cases of harbor improvement on our British coasts. The absence of tidal and littoral currents is in itself a distinctive feature of the harbors to which Captain Black refers. In many of our British ports these form such an important factor that they cannot be disregarded. In the case of the port of Dublin, with which I have been

* "The Improvement of Harbors on the South Atlantic Coast of the United States." By William Murray Black, Captain, Corps of Engineers, U. S. A., M. Am. Soc. C. E. *Transactions*, Vol. XXIX, page 223.

† Of Dublin, Ireland.

associated for the last twenty years, the keynote of the success which has attended the efforts made to improve the bar has been due to a full appreciation of these littoral and tidal currents by the engineers who designed the works which led to the great increase in the depth of water on Dublin bar.

Dublin bar has been improved by artificial scour resulting from the construction of a tidal reservoir of 2 500 acres, enclosed by two great piers extending out over the strands of Dublin Bay. These piers have been so formed as to make the outward scouring current flow in unison with, and not in opposition to, the littoral currents in the bay. Before beginning the works which have proved so successful, both time and money were spent in fully investigating the problem. I have fully described these works in a paper read before the Institution of Civil Engineers in 1879, which is published in their Proceedings, and it is therefore unnecessary for me to enter now into greater details than to say that they have resulted in increased depth on the bar of 9 ft. 6 ins.

I most cordially agree with Captain Black in a point to which he refers more than once, namely, the injurious effect of "meager and intermittent supply of money for carrying works into execution." In marine works this seems to be specially the case. The sea seems almost to take a delight in thwarting a timorous or vacillating policy. I could name numerous works which have suffered from this. The lay mind of the Parliamentary representative, who has to vote the funds for such works, readily falls in with the argument that it is better to make a comparatively small annual grant instead of voting the full estimated cost, on the ground that they will see how the work succeeds before voting away large sums of money, the result often being that extraneous influences are brought to bear, votes are cut down, works are delayed and much of their utility lost by encroachment and damage to the unfinished work. In the case of piers run out on sandy shores, slow progress is often followed by rapid advancement of the low-water line.

The manner in which the works for the improvement of Dublin bar were carried out forms a striking exception to this hesitating policy. The Dublin Harbor Board, then known as the Ballast Board, acted very wisely in first obtaining the best professional advice, and thoroughly investigating the problem, and then obtaining the necessary

funds before proceeding with the work. By this means the works were allowed to proceed rapidly, and I believe this proved to be no small factor in their success. In fact, I believe the only way to deal with these problems is, to investigate the case thoroughly, to make up your mind thoroughly, and to proceed with the work rapidly. It is better to proceed thus than to endeavor to find the best possible design by a series of tentative stages.

The introduction of sand-pump dredgers in England seems to promise great assistance in dealing with ports on sandy coasts. A very interesting experiment is at present being carried out on the Liverpool bar, where they have just got a sand-pump dredger designed to raise 3 000 tons per hour. As the result of two years' dredging with their smaller sand-pump dredgers I was able to cross the bar in the R. M. S. *Britannic* a few weeks since at one hour after low water, which was unique in the experience of the White Star Line. We are therefore watching the result of this great dredging experiment with much interest.

William Murray Black, M. Am. Soc. C. E.—The method of conducting the improvement of rivers has been alluded to, and, I think, in justice to the branch of the service to which I belong and by which the greater part of the work has been done, it may be well to explain how it comes that the work of river improvement is so unsystematic as it is. The award of appropriations for the improvement of rivers and harbors lies in the hands of Congress. A number of years ago it was enacted that no improvements should be started until the necessity for them had been certified to by the Secretary of War.

The routine method is this: In the River and Harbor bill is inserted provisions like the following, taken from the act approved July 13th, 1892. * * * "The Secretary of War is hereby directed to cause preliminary examinations to be made at the following localities, to wit:" * * * (Here follows a list of the points to be examined.) * * * "That the preliminary examinations ordered in this Act shall be made by the local engineer in charge of the district, or an engineer detailed for this purpose; and such local or detailed engineer and the division engineer of the locality shall report to the Chief of Engineers, first, whether, in their opinion the harbor or river under examination is worthy of improvement by the General Government, and shall state in such report fully and particularly the facts and reasons on which they base such opinion, including the present and prospective demands of commerce; and, second, if worthy of im-

provement by the General Government, what it will cost to survey the same with the view to submitting plan and estimate for its improvement; and the Chief of Engineers shall submit to the Secretary of War the reports of the local and division engineers, with his views thereon and his opinion of the public necessity or convenience to be subserved by the proposed improvement ; and all such reports of preliminary examinations, with such recommendations as he may see proper to make, shall be transmitted by the Secretary of War to the House of Representatives, and are hereby ordered to be printed when so made." Then follows the appropriation of a lump sum to cover the cost of the work so directed.

The money so appropriated is allotted to the various works, and the examinations are made and the reports submitted as required. The examinations and surveys are made as carefully and fully as the allotments will permit. The funds available are not always as great as the local engineers require for the best results. But insufficient appropriations seem unavoidable when the extent of our country and the great number of local demands for improvement are considered. Indeed it seems that we are not yet sufficiently wise to appreciate the absolute interdependence of the various portions of our country and to see that the prosperity of all is dependent on the prosperity of each. Each community, while seeing and urging the claims of its own locality, forgets or neglects the needs of others, and becomes indignant at the inadequate provision for its local improvements, while holding up its hands in horror and calling names on account of the large sums appropriated for such work in general. The pressure of public opinion compels Congress to provide for works of improvement at many points, while the same pressure prevents Congress from making the adequate appropriations which, as is fully known, the interests of commerce and of true economy both demand.

In preparing his project the local engineer considers to the best of his ability all of the factors involved. Important projects are then generally submitted to a board of engineers for further consideration, and finally they are considered by the committees of the two Houses of Congress. It is a grave mistake to assume that this final consideration by Congress is simply a log-rolling affair. The writer has appeared at times before the committee of each House, and has found the members, not only interested in the engineering points on which he was interrogated, but posted to an unlooked-for degree on the local peculiarities of the works of his district.

An instance of the injustice of the popular clamor about log-rolling in connection with the River and Harbor bills is given in the history of the Act of August 2d, 1882. This act was vetoed by President Arthur, after many accusations of log rolling and extravagance had been made by a portion of the public press. It was passed over the President's veto. At the following session, the House of Representatives called on the Secretary of War by resolution for a list of the appropriations made in the Bill for Improvements which he considered unworthy, with his opinions. When this list was given to the House it was found that the total amount of the appropriations objected to was only 5½% of the total amount appropriated in the bill and a subsequent investigation showed that the objections urged with regard to many of the objected-to items were based on a misconception of the facts.

Should Congress consider the work proper and the project judicious, it then starts the improvement by an appropriation. Here, again, the vast number of demands hampers it, and the appropriation necessarily is insufficient. In the case of a few of the more important works Congress has made provision for continuous work by special appropriations; but this, in the present state of public opinion, is manifestly impossible for the great number of works now being carried on, nor is it possible, for the same reason, to abandon any of the partly completed works.

The results of insufficient appropriations are twofold. The works cannot be carried on in an economical manner, nor can the work be done systematically. The same local demand which calls for the widely distributed appropriations calls for the immediate expenditure of the sum available, if only for the removal temporarily of the most annoying obstructions to navigation. Congress and the engineers are alike servants of the people and must do their will. That the projects are as well considered as human failings will permit, and that the engineers, executive and advisory, are free from local prejudice and are earnest in the discharge of their duties, will, I think, be admitted. The shortcomings of the system can usually be traced to the demands of the people, who pay for their work, and the remedy lies in that broader education which will admit the judgment of a wider knowledge and which will hold local interests subservient to the needs of the whole nation.

W. P. Craighill, M. Am. Soc. of C. E.*—The thoughtful reader of Captain Black's very valuable paper will find profit as well as interest in examining very carefully an article on sea beaches by Professor H. S. Shaler, of Harvard College, in which he treats that subject in detail and in a most instructive manner.

Captain Black is, I think, quite right in his supposition that the mouth of the Cape Fear River was at some time near Federal Point. It is even probable that the mouth in the more distant past was still nearer Wilmington. The existence and character of a peculiar obstruction called "the logs," formerly found a few miles below Wilmington, seem to prove it. This obstruction was called "the logs" because supposed to consist of sunken logs on the bottom which had floated down from above and been lodged there. When this impediment to navigation was attacked in the progress of the improvement of this river under my direction, no logs were found at all. It consisted of the stumps of a cypress forest which had evidently stood long years before. These stumps were found *in situ* in what was the main channel of the river at the time of their removal. Their tops were, many of them, nearly as smooth as if they had been sawn across, and all at about the same level of 9 ft. below mean low water. The machine used in the first removal of the stumps was a clam-shell dredge from which the bucket was removed and replaced by a grapnel. Generally the power of the engine was sufficient to pull the stumps, with attached roots, etc. In some cases it was necessary to depend on the slow but irresistible lifting action of the rising tide. The stumps were sound, and this condition might have come from their immersion in the mud which was even with their tops. But the trees had disappeared. How and why had they been cut off? It was supposed the land had at some time subsided so as to permit the income of the teredo which had eaten off the trees and then destroyed them completely after their fall into the water. But this theory required a rising again of this same portion of the soil, as the water there does not now contain the teredo or any insect destructive to timber.

The place called the "Haulover," where the New Inlet was opened in 1761, took its name from the habit of boatmen who hauled their craft over the lowest part of the low sand beach which separated the ocean from the river. The beach was gradually worn down at this point, and was in a condition to be more readily broken through when the gale came in 1761 to open the inlet.

* Col., Corps of Engs., U. S. A.

Since the closure of New Inlet, while the accumulation of sand on the actual line of the closure has been very small, the beach has grown and extended very much to the southwest from a point nearly opposite Fort Fisher, so as to form a large bay between it and the closure-dam. This bay will probably fill in time with sand which is thrown up on the spit by the waves, and then dried and blown by the wind into the bay.

The day will probably come when people seeing a wide expanse of sand between the ocean and the present dam will wonder why such a dam was built, as the need for it at any time will then not be apparent. And it is not unlikely the material from it may some day be safely removed and used at other points, as, for instance, in constructing one or more jetties at the mouth of the river, should it ever become necessary to greatly increase the depth now available there.

The space where General Butler's powder-boat was exploded off Fort Fisher, and where the fleet lay during its attack and capture on a subsequent occasion, is now filled with sand and is either dry at low water or nearly so. Zeke's Island has also grown very much since the building of the closure-dam.

The shape of the closure-dam from Federal Point to Zeke's Island has been criticized and inquiry made why it was not in a straight rather than a broken line. The reason for the peculiar shape is the following: When the dam was planned there was a long shoal extending in a southwest direction from Federal Point, and one then extending in a northwest direction from the upper end of Zeke's Island. The sections of the dam from Federal Point and Zeke's Island were placed on these shoals, because they were there and because the section next Federal Point would act as a deflector to the ebb current as it came down the river, throwing some of it at least into the main river and away from the inlet. The section of the dam next Zeke's Island was placed so as to have a similar action on the flood current.

Much scour took place because of insufficient funds to push the work as rapidly as it should have been. This was particularly the case when the work was about the height of low water and between that level and that of high water. The distance from the dam to the ocean by the way of New Inlet being much shorter than by the proper mouth of the river, the tide changed its level on the ocean side of the dam much more rapidly than on the river side. This produced a very

strong tidal current at the dam, which increased in velocity as the opening was lessened. After the dam had reached the level of low water, there was quite an overfall at times, which also produced scour.

From necessity, much of the dam was built of small rock procured from the Cape Fear, the cost of large rock from the north or the interior prohibiting its use to any great extent. The unfinished dam of these small rocks was more than once much degraded by easterly gales which brought in the waves from the ocean with great violence.

The dam was finally coped with large flat rock. Some of these coping-stones are still occasionally displaced by severe storms.

The use of the "log-mattress" was due to economical considerations. This mattress was constructed in a very simple way of the materials at hand. The pine trees on the creeks emptying into the river near by were cut down and their trunks rolled into the stream and formed into rafts. These rafts were loaded with as much of the brush of the same trees as they would carry. They were then towed into position and sunk by being loaded with stone thrown upon them from barges. The prolongation of the dam below Zeke's Island was founded, as Captain Black states, on more regular mattresses, one of the continuous mattresses being three-fifths of a mile long, and very successfully placed.

While the jetty constructed many years ago at Baldhead Point with the object of holding that headland was a complete success from that point of view, it is worthy of note that the depth of the Baldhead Channel began to increase at the same time the jetty was separated from the shore. The question arises whether a jetty in that position was not injurious to the channel. The spur jetties constructed many years ago on the Fort Caswell side of the Baldhead entrance were completely successful in protecting the walls of that fort, which were once seriously threatened by the ocean.

It is considered that the beneficial effect of the work of the pump-dredge Woodbury on the Baldhead Channel was much greater than merely the removal of the cubic yards taken into her bins. Much more material was loosened by her drags and carried by the ebb tide into the deep water beyond the bar. Her work on the bar was mainly in a straight channel on ranges, the object being to localize the action of the increasing current as New Inlet was being closed, and to facilitate the deepening within moderate horizontal limits rather than have it spread out over the whole bar.

Much value should be given to the opinion of Mr. Bacon, quoted by Captain Black, that he was "convinced" from his observations that "the foundation, placed on sand or mud, most secure from undermining by water, is of small rip-rap stone," for Mr. Bacon was a man of most excellent judgment and a close observer. But it is thought this statement is too general. It is true that certain experiments, made by Mr. Bacon by my direction, showed that the mattress could be safely dispensed with in certain parts of the dam below Zeke's Island where the soil was of a special character, but it must be borne in mind that the location of that part of the dam was comparatively sheltered from the action of waves or currents. It is believed the mattress foundation can seldom be dispensed with where the underlying mud is soft and deep, or the sand is exposed to erosion.

Captain Black characterizes the work on the Cape Fear as "successful," and he is right (for it is a success to procure 18 ft. at low water in a great river where 9 ft. had been the ruling depth), and it gives me great pleasure here to record my obligations to Mr. Bacon who was my co-worker on it for many years. To him more than to any other man is due the success of the work. When he died of the grip a few years ago, I lost a good and trusted friend, and the General Government a most valuable and faithful servant. An honest, true man went to his rest from unremitting labor fruitful in good results.

Captain Thomas W. Symons.*—I have read with great interest Captain Black's account of what is going on in the way of harbor construction on the South Atlantic Coast. Such a paper gives to us who are personally unfamiliar with this section a clear and succinct account of what is going on there, and how plans, methods and ideas differ from those developed on this western coast. On the South Atlantic, as well as on the Pacific, there seems to have been an elimination of certain methods, processes, and devices, and a gradual settling down to an adopted style of work, with which Captain Black appears to be satisfied.

From Captain Black's description, it is evident that the works at the mouth of the Cape Fear River, at Charleston, and at St. Johns, are so widely at variance with similar works on the Pacific Coast as to be a matter of mark. It is safe to say that no officer having experience with similar work on the Pacific Coast would be satisfied with the plans and methods adopted and pursued at these Atlantic Coast ports. Starting

*Capt., Corps of Engrs. U. S. A.

from the sheet piling work at Wilmington, Cal., and the low-tide crib work at Yaquina, the officers of the Corps of Engineers on the Pacific Coast have gone through all the stages, and have now arrived at the conclusion that the proper form of jetty construction for the improvement of bar harbors is a high-tide stone jetty on a thick brush mattress foundation, built from a pile tramway, as described by me in a recent paper on the jetty harbors of the Pacific Coast. This is very different from the thin log or brush mattresses with low-tide rubble mounds, all built from floating plant, as described by Captain Black.

The Pacific Coast or tramway system presents many marked advantages over the Atlantic Coast system. It furnishes a solid and substantial base on which mattresses of any desired thickness and width can be readily made at the site they are to occupy, and where they can be quickly sunk into place and held there by the piles and the stone placed upon them. All operations can be carried on continuously regardless of the tide, and almost regardless of storms. It is applicable in the shallowest tide flats, and in the deepest water where operations are liable to be carried on. The last 1 000 ft. of the Coos Bay jetty tramway was built in water 26 ft. deep at low water, with piles 80 ft. long, and which penetrated 30 ft. into the sand. It is to be noted that at Galveston the tramway system was adopted after years of trial of other devices.

The swells and waves of .the Pacific, and especially along the coast of Oregon and Washington, would, it is believed, render it impossible to carry on the work of jetty construction as it is carried on at Charleston and the other points named by Captain Black. Out here, there probably would be not to exceed twenty days in a year when the method of towing out and sinking mattresses described by Captain Black (*Transactions*, Vol. XXIX, pages 266 and 267) could be successfully carried out, and these days would be bunched in the summer.

I am inclined to believe that the sand movements on the Atlantic Coast are not nearly equal in magnitude and rapidity to the sand movements on the Pacific Coast. On the Atlantic Coast, there seems to be a decided general resultant sand movement to the south; on the Pacific there is no such general resultant movement, or, if there is, it is so modified by local circumstances as to be indistinguishable.

The supporting power of the sand appears greater on the Pacific Coast than on the Atlantic. So great is this supporting power that

some of our ablest engineers question the necessity of putting any mattresses down for the support of the rubble mounds. It is probable that the South Atlantic sands are finer and lighter than are those of the Pacific, thus necessitating the very wide mattresses described.

On the other hand, the channels naturally developed through the bar sands on the Atlantic Coast appear to have a far greater persistence than have similar channels through the bar sands of the Pacific.

The important function of jetties to impound the moving sands does not seem to be recognized on the Atlantic Coast with the force that it is here. This is particularly evidenced by the plan adopted at Charleston, where, instead of impounding it outside of both jetties, it is proposed to keep it moving to the south across the inner ends of the jetties, where the width between them is greatest. I should fear that such a condition would result in the formation of an inner bar or shoal, materially limiting the capacity of the harbor.

The work at the Cape Fear River appears to have been eminently successful. I should judge that the entrance to the river received very valuable protection against storms from the north and east, from Smith's Island and the Frying Pan Shoals, and that the maintenance of the good entrance by dredging would be comparatively easy. One of the lessons learned in this work was also taught by our Pacific Coast works, *i. e.*, that it is better to build up gradually from the bottom over a long line than to push the completed structure bodily to the front.

The work at Charleston is certainly interesting as to plan, methods, and results. It is evident, however, that the work is far from completed. The wisdom of leaving the jetties low near their inner ends may well be questioned, and it was well, in formulating the plan, to leave this height as well as the length of the jetties problematical. It looks to me as if the adopted width between the ends, 2 900 ft., would require the entire tidal flow to pass through to secure and maintain the deep-water channel aimed at. This will necessitate building both jetties up to full high tide.

The mattresses of logs have, as pointed out by Captain Black, the great demerit of comparative inflexibility. Experience on the Pacific Coast would indicate that a better construction would have been a mattress, 3, 4, or 5 ft. thick, of brush fascines and poles, well tied and wired together.

At St. Johns, a study of the comparative charts and a knowledge of the resultant movement of sands and channels to the southward rather indicates that it would have been wise to adopt the southern entrance channel of 1879 as the one to be improved. From the comparative charts and profiles and Captain Black's statements of the results accomplished, it is evident to me that it will require the entire tidal flow to pass in and out between the jetties, to develop and maintain the maximum channel over the bar with the adopted width between ends of jetties.

The existence of the two channels branching near the ends of the jetties is indicative of the fact that there is not at present a proper co-relation between the tidal flows, the active forces and the width and height of the jetties built to control and direct these forces.

Captain BLACK.—There are a few statements made by Captain Symons which I think will bear further discussion. Captain Symons names the method of jetty construction from tramways "The Pacific Coast System," and that from floating plant "The Atlantic Coast System," it is presumable, simply as a convenient mode of distinguishing them apart. Neither system was devised or used exclusively on either coast. Both systems have been used for many years abroad, and both are in use on the Atlantic Coast. The tramway method has been applied on the greatest scale at the mouth of the Columbia River, Oregon, and there its details have been most skillfully elaborated.

The decision as to which system is best adapted for any particular work must be based on the question of relative economy. The tramway system is certainly best for stormy sites, and where the available funds are sufficient in amount, and reasonably continuous in supply. Where the amounts available are small, and the supply intermittent, it is manifestly advantageous to adopt a system in which the capital sunk in destructible and non-removable plant is as small as possible. The improvements described in the paper were started in the days of small and very irregular appropriations, and that is doubtless the reason for the construction methods adopted. Later, when large and more regular appropriations became available, the adoption of the tramway method was found to involve too great an outlay to be advisable at the stage to which the works had been advanced, under the conditions of wind and weather prevailing.

The wide divergence of the completed jetties in type, which Captain Symons finds, is not evident. On both coasts, the jetties are of rubble, as large as can be handled economically, on a mattress foundation, and are built to the height necessary to control the required volume of flow, and no higher. In all cases the height is determined by the conditions at the particular work. Captain Symons' statement as to the results of the experience of the engineers of the Pacific Coast would seem to be somewhat sweeping. It is hardly likely that anyone would admit that his mind is of so inflexible a texture that it must insist on one particular type of work regardless of local conditions.

He is also somewhat contradictory regarding the value of thick mattresses. In one paragraph he states that the "conclusion arrived at concerning the proper type of jetty calls for a "thick brush mattress foundation." Elsewhere he states that "some of our ablest engineers question the necessity of putting any mattress down for the support of the rubble mounds." From this it appears that, after all, the mattress is regarded rather as a necessary evil. It has been the experience of the writer, that where not covered promptly by sand or silt, the mattress was gradually destroyed by the teredo. Where any great thickness was used in a jetty, the destruction of a portion of the wood, and the compression of the remainder, caused a gradual subsidence of the jetty crest, and necessitated a continued building up with stone. For these reasons he prefers a tightly compressed mattress as thin as consistent with the requisite degree of strength and impermeability.

The use of a jetty for impounding sand is also of doubtful value where large volumes of sand are moving. It is a simple impossibility to stop the entire sand movement of the Atlantic Coast for any prolonged period. If the sand cannot be induced to move past the jetties without blocking the channel, the necessity of further jetty extension is simply a question of time, unless dredging be resorted to, as abroad.

The Charleston and St. Johns works are not yet completed, and therefore no conclusions can be drawn from them definitely, no criticism on the height of jetty is possible, for no height has as yet been fixed upon. The latest advices from both works show that the improvement of the jetty channels has continued since the paper was written. At Charleston there is now a straight channel from the sea to the city 15 ft. deep at low water, and 18 ft. deep at low water for all

but 1 000 ft. At the St. Johns the channel has also greatly improved. The forking of the channels at the entrance of the St. Johns jetties was the necessary result of the rolling out of the immense volume of sand which lay between the jetty sites, the deeper channels forming where the jetties were most affective, close alongside.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

662.

(Vol. XXX.—December, 1893.)

RAILWAY TERMINAL SYSTEMS.—DISCUSSION ON PAPER No. 612.*

By C. O. GLEIM.

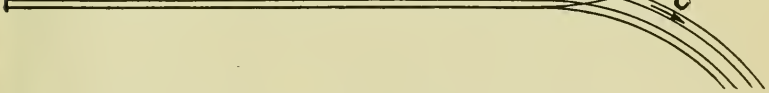
C. O. GLEIM, Cor. M. Am. Soc. C. E.—In revising the remarks made by me orally at the International Engineering Congress, I suppress so much of them as referred to some misunderstandings contained in the advance copies of the paper, which have now been corrected.

The new terminal system for the sister cities of Hamburg and Altona, with the Hamburg part of which I have been officially connected for several years, will form a very interesting case of terminal arrangements for large cities. The change at present contemplated, and partly executed, is the consequence of the absorption by the Prussian Government of all the railways entering the two cities, with the one exception of the Lubeck line. The various lines entering on the Hamburg side will be joined in a central station, from which the trains will pass on, over the connecting railway, through three intermediate stations, to the term-

* "The Rearrangement of the Railway Terminal System at Altona, with Special Reference to the Avoidance of Grade Crossings." By R. Caesar, Member of the Royal Board of Railroad Directors at Altona, Prussia. *Transactions*, Vol. XXIX, page 295.

inal at Altona, while the trains entering on the Altona side are to pass over the same connecting railway, using the same intermediate stations, and terminating in the Central Station of Hamburg. The overlapping system thus formed will be very similar to that of the existing Metropolitan Railway (Stadtbahn) of Berlin. The same similarity will exist in the carrying through, in both cases, of four independent railway tracks, of which two will be for city traffic, and two for through passenger traffic and freight. The connecting railway between Hamburg and Altona, which is now a double-track line, will at the same time be raised so as to abolish present grade crossings of streets.

In carrying out the system thus outlined, a complication arose on the Altona side, which forms the subject of the paper. Local influences caused the terminal station of that city to be retained as near as possible to its old position, and this necessitated, instead of an ordinary through station, the arrangement of a "pocket station," or, as it is termed in Germany, a "head station," where the trains reverse their direction, and go out at the same end from which they entered. A station of this kind necessitates inconvenient crossings of main tracks already when forming an intermediate station of a merely double-track line. An arrangement like the following diagram, with one track crossing, would not be sufficient; for the train coming in on the right-hand track from *A*, and reversing on the track *B*, would reach the left-hand track *C*, as shown by the arrows, and another crossing would be required to make it reach the right-hand track near *C*. How much



more complicated the crossings become in the case of a four-track line, as at Altona, is shown in the paper. Fortunately, it was possible, without exceeding grades of 1 in 100, to raise and lower the tracks so as to arrange a system of over and under grade bridges, and avoid practically all grade crossings between main tracks. Although the well-known example of the Cannon Street Station in London as an intermediate "pocket station" between London Bridge and Charing Cross, has proved that such a station can, even under a very heavy

traffic, be worked with level crossings of the tracks, if protected by a well-regulated signal system, there can be no doubt that, where practicable, a solution based on passing the tracks by bridges over one another, as at Altona, is to be preferred. Perhaps it was a mistake in this case to have given way to local interests so far as to accept a situation precluding the arrangement of a through station; but the "pocket station" having once been accepted, the system, with the avoidance of grade crossings, has been carried through in the most consistent way, and Plate I (Vol. XXIX, page 296) is well worthy of a thorough study. If drawing is the universal language of the engineer, requiring no translation, this drawing will give a better idea of the work treated in the paper than any lengthy description could do.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

663.

(Vol. XXX.—December, 1893.)

LOADINGS FOR RAILWAY BRIDGES.—DISCUSSION ON PAPER No. 618.*

By WARD BALDWIN, J. A. L. WADDELL and C. D. PURDON.

WARD BALDWIN, M. Am. Soc. C. E.—The loading incorporated in a set of railway bridge specifications is usually based upon the weights and wheel spacings of the heaviest engines and cars used or likely to be used on the particular railway. A sufficiently extended list of the loadings now specified for railway bridges may therefore be regarded as a reflection of the combined judgment of railway engineers as to what loadings should be specified to meet their needs; and a study of such loadings would appear to promise more fruitful results than a study of actual locomotive loads. The systematic efforts of J. A. L. Waddell, M. Am. Soc. C. E., to secure the adoption of a set of standard loadings for railway bridges, have directed special attention quite recently to this subject, and the data in Mr. Purdon's paper are of interest as they bear directly upon this question; but, so far as known to the writer, a classified compilation of specified loadings, although doubtless often made, has not yet been published. Such a compilation would appear

* "Comparison of Modern Engine-Loading with Standard Specifications for Spans from 10 to 200 Feet." By C. D. Purdon, M. Am. Soc. C. E. *Transactions*, Vol. XXIX, page 426.

to be the necessary preliminary to an intelligent discussion of standard loadings, and the writer therefore ventures to add to Mr. Purdon's statistics as complete a list of bridge loadings as he has been able to collect. It is thought this list will be valuable as a record of the present opinions as to proper bridge loadings, and will furnish data for some interesting deductions. This list will also show that the complaints of the bridge computers against needless multiplication of loadings are not without cause.

A concentrated loading enables the railway engineer to secure the proportioning of his bridges to sustain, as near as theory can decide, the exact loads that will come upon them. It is this fact that has led to the general use of concentrated loadings by railway engineers, rather than any desire on their part to be ultra theoretical. To this fact also must be ascribed the much-complained-of diversity of loadings, because the attention of the railway engineer has been directed rather to the determination of a loading suitable for his road, than of one that would make the work of the bridge computer easy. This method adopted by the railway engineer for selecting bridge loadings is undoubtedly the only rational one, and will not, therefore, be abandoned simply because it increases the labor of bridge computers. But there is no advantage in adhering strictly to the exact wheel weights and spacings of the actual loadings, as is done for example in loading 90 of the table. The real distribution of the loading is seldom, if ever, the same as found by weighing the wheel weights when the train is at rest; and the stresses produced by the actual loading would, so far as can now be known, be as near those given by a typical loading with wheel spacings in even feet, and wheel loads to the nearest 1 000 pounds, as those given by a loading with dimensions more minutely agreeing with actual static dimensions. This consideration would seem to show that it would be no sacrifice of accuracy to reduce unnecessary drudgery in computations by rounding off the figures. And other facts, such as differences in allowed unit stresses, lead to the conclusion that substantial accuracy in results may be secured so long as the general distribution of the actual loading is observed in the typical loading, even though the latter may differ considerably from the actual load in detail. Thus it would seem reasonable to expect that a comparatively small number of standard loadings might replace the large number of loadings in use at present. That a

great saving of labor for the bridge computers would result from the general adoption of a set of standard loadings is evidenced by the variety of loadings given in the list above; and there is no question but that such loadings would be readily recognized as standard, provided they were so selected as to cover substantially the same ground as the present specified bridge loadings.

The above list, comprising the loadings now specified by one hundred independent railway systems of the United States and Canada, seems comprehensive enough to serve as a basis for a study of at least some of the conditions a set of standard loadings should fulfill. In the list, the loadings used by five of the Canadian railways (Nos. 79 to 83 of the list) are included. All Canadian railways that receive government aid, and most of them do, are required to use this loading, which is that contained in the general specifications for railway bridges issued by the Chief Engineer of the Government railways of Canada; and this loading may therefore be considered as the established standard for Canadian railways at this time. These five railways will therefore be disregarded in the succeeding remarks, and attention will be confined to the 95 remaining railways, which are all in the United States. Among these 95 railways, 10 (Nos. 91 to 100 of the list) specify some kind of uniform load with, in some cases, additional excess loads. As the actual distributions of the corresponding concentrated loading from which these uniform loadings have been derived are not given, these 10 loadings will also be omitted in the present inquiry. Among the 85 remaining railways—

71 specify only one kind of loading.
10 specify two kinds of loading.
4 specify three kinds of loading.
82 specify a loading made up wholly or in part of consolidation engines.
12 specify a loading made up wholly or in part of passenger engines.
5 specify a loading made up wholly or in part of decapod engines.
3 specify a loading made up wholly or in part of ten-wheel engines.
1 specifies a loading made up in part of mogul engines.
3 require a uniform load to precede as well as follow the engines.
82 do not require any load to precede the engines.

Among the 82 railways that specify a loading made up wholly or in part of consolidation engines—

4 vary the weights on the drivers.
78 do not vary the weights on the drivers.
9 vary the weights on the tender wheels.
73 do not vary the weights on the tender wheels.

77 specify two consolidation engines and a uniform load.
3 specify a train of consolidation engines.
1 specifies two consolidation engines and wheel-loads for train.
1 specifies the 4 driver-loads of a consolidation engine, and a uniform load.

In the above enumeration, railways 20 and 34 of the list are counted twice, because they specify two loadings, each consisting of two consolidation engines and a uniform load.

Of the 77 loadings of two consolidation engines and a uniform load, there are—

27 that are each used by 1 railway.
4 " " 2 railways.
3 " " 3 "
2 that are each used by 4 railways (Cooper's "Lehigh Valley" and P. R.R. specifications).
1 that is used by 9 railways (Cooper's Class "A").
1 " 16 " (Cooper's Extra Heavy "A").

These figures make it clear that the only loadings that have as yet been at all generally adopted are those given in Mr. Cooper's bridge specifications.

Out of these same 77 loadings—

1	requires a	22 000-lb.	driver load	with a	2 240-lb.	uniform	load.
1	"	23 000	"	"	3 500	"	"
12	"	24 000	"	"	3 000	"	"
1	"	24 640	"	"	3 000	"	"
3	"	28 000	"	"	3 000	"	"
1	"	28 000	"	"	3 500	"	"
1	"	28 600	"	"	4 000	"	"
18	"	30 000	"	"	3 000	"	"
1	"	30 000	"	"	3 500	"	"
14	"	30 000	"	"	4 000	"	"
1	"	30 000	"	"	4 200	"	"

1	requires a	31 000-lb.	driver-load	with a	4 000-lb.	uniform	load.
1	"	32 000	"	"	3 600	"	"
2	"	32 500	"	"	3 500	"	"
1	"	34 000	"	"	3 000	"	"
1	"	35 000	"	"	3 500	"	"
4	"	35 000	"	"	4 000	"	"
1	"	36 000	"	"	4 000	"	"
2	"	average 37 000	"	"	4 000	"	"
6	"	40 000	"	"	4 000	"	"
1	"	44 000	"	"	3 200	"	"
1	"	average 36 600	"	"	3 000	"	"
2	"	only gross weights of engines			3 000 4 200	" "	" "

1	specifies a	uniform	load of	2 240	lbs. per ft.
37	"	"	"	3 000	"
1	"	"	"	3 200	"
6	"	"	"	3 500	"
1	"	"	"	3 600	"
29	"	"	"	4 000	"
2	"	"	"	4 200	"

If the 85 loadings that form the basis of this paper can be accepted as fairly representative of the loadings that are needed by railway engineers, the conclusions that the comparisons given above seem to justify are :

1. A loading consisting of two consolidation engines followed by a uniform train-load should be the standard form of specified loading; but this should be supplemented by a heavy passenger engine for short panels.

2. In a set of standard loadings the uniform load should vary by increments of 250 lbs, from 3 000 lbs. to 4 250 lbs. per foot.

3. The driver-loads should vary by increments of 3 000 lbs. from 24 000 to 36 000 lbs.; and by increments of 4 000 lbs. from 36 000 to 44 000 lbs. The tender-wheel loads should vary by increments of 1 000 lbs. from 17 000 to 23 000 lbs. The truck-wheel loads should vary by increments of 1 000 lbs., from 14 000 to 20 000 lbs.

4. The driver and tender wheels in a given loading should be given uniform weights.

The difficulties in the way of arriving at a suitable set of standard engine-loadings do not seem to be very great. A large majority of the railways now specify consolidation engines, and it only remains to secure an agreement as to certain standard wheel-spacings and graded wheel-loads. A problem of more difficulty, perhaps, is the selection of the uniform train-load to be coupled to a given engine-load, because the uniform load now used by many railways does not seem to bear any general relation to the engine-loading. Mr. Purdon has called attention to the fact that the Pennsylvania Railroad loading, consisting of two consolidation engines, followed by a uniform train-load, produces smaller chord stresses in a long-span bridge than the train-load with the engines off the span. In this loading, the aggregate engine-loads, if distributed uniformly over the engine-wheel base, would be only about 3 700 lbs. per foot, while the train-load is 4 000 lbs. per foot. A case of the extreme opposite ratio of engine-load to train-load is the loading for the Atchison, Topeka and Sante Fé Railroad, in which case the aggregate engine-loading uniformly distributed over the engine-wheel base would be about 5 200 lbs. per foot, while the train-load is only 3 200 lbs. per foot. It will be readily perceived from an inspection of the list of loadings given above that in many cases the heavier engines are attached to the lighter trains, and *vice versa*. Thus it would appear that each uniform load should be coupled to several of the engine-loads somewhat as follows:

The 3 000-lb. uniform load to each of the engines having from 24 000 to 36 000-lb. driver-loads, and each of the other uniform load from 3 250 to 4 250 lbs. per foot, to each of the engines having from 30 000 to 44 000-lb. driver-loads.

Using the wheel spacing given by Mr. Waddell on Plate XVI, *Transactions*, Vol. XXVI, p. 90, the set of loadings resulting from the above combinations would be (see table on page 521).

The tenders for the passenger engines are the same as for the corresponding consolidation engines. The engine-loads in this set of loadings do not differ much from those proposed by Mr. Waddell, but they agree somewhat more closely with the driver weights that are used in current specifications. The plan of coupling one engine-load to several uniform loads is a radical departure from the plan adopted by Mr. Waddell, of coupling each engine-load to a single uniform load, the uniform load increasing regularly with the driver-load of the engines coupled to it. It does not seem possible to devise a system of

Two Consolidation Engines.															Uniform load.	Passenger Engine.						
8′	5′	5′	5′	8′	5′	5′	5′	8′	.....	4′	.....	6′	8′	7′	8′	Tender.
14	24	24	24	24	17	17	17	17	14	2d engine	17	3 000	20	20	40	40	17
8′	5′	5′	5′	8′	5′	5′	5′	8′	.....	4′	.....	6′	8′	7′	8′	"
15	27	27	27	27	18	18	18	18	15	2d engine	18	3 000	20	20	41	41	18
8′	5′	5′	5′	8′	5′	5′	5′	8′	.....	4′	3 000 3 250 3 500 3 750 4 000 4 250	6′	8′	7′	8′	"
16	30	30	30	30	19	19	19	19	16	2d engine	19	....	20	20	42	42	19
8′	5′	5′	5′	8′	5′	5′	5′	8′	.....	4′	3 000 3 250 3 500 3 750 4 000 4 250	6′	8′	7′	8′	"
17	33	33	33	33	20	20	20	20	17	2d engine	20	....	20	20	44	44	20
8′	5′	5′	5′	8′	5′	5′	5′	8′	.....	4′	3 000 3 250 3 500 3 750 4 000 4 250	6′	8′	7′	8′	"
18	36	36	36	36	21	21	21	21	18	2d engine	21	....	20	20	46	46	21
8′	5′	5′	5′	8′	5′	5′	5′	8′	.....	4′	3 250 3 500 3 750 4 000 4 250	6′	8′	7′	8′	"
19	40	40	40	40	22	22	22	22	19	2d engine	22	....	20	20	48	48	22
8′	5′	5′	5′	8′	5′	5′	5′	8′	.....	4′	3 250 3 500 3 750 4 000 4 250	6′	8′	7′	8′	"
20	44	44	44	44	23	23	23	23	20	2d engine	23	....	20	20	50	50	23

NOTE.—The wheel loads are given in 1 000 pounds units.

loadings on this latter plan that will agree with the conditions of actual traffic. For example, consolidation engines having driver-loads of from 24 000 to 36 000 lbs. are now in use on many railways, where the heaviest cars hauled are of 60 000 lbs. capacity, weighing about 2 650 lbs. per foot, so that present traffic conditions seem to require that the uniform load of 3 000 lbs. should be coupled with each engine-load up to those having at least as much as 36 000-lb. driver-loads. This seems the more necessary when it is remembered that the uniform load of 3 000 lbs. is about 12% in excess of the weight of a train of loaded cars of 60 000 lbs. capacity, and, therefore, represents a load seldom reached, except on roads carrying ore trains, while engines having as high as 36 000-lb. driver-loads are in common use to-day. While the coupling of several engine-loads to each uniform load multiplies the number of loadings, still this is a matter of no great moment, for the most important advantage to be gained from a set of standard loadings, viz., of restricting the types of engine-loadings to a small number and simple dimensions, would be attained by using such a set of loadings as are given above.

The suggestion is offered, that in specified loadings where mogul or 10-wheel engines are used, consolidation engines with equal driver-loads may be properly substituted; and where decapod engines are used, consolidation engines with equal aggregate weight may be substituted, if a set of standard loadings made up of two consolidation engines and a uniform load should be adopted. The writer ventures to express the hope that the Society will regard this problem of sufficient importance, and its satisfactory solution promising enough to warrant its consideration by a special committee.

J. A. L. Waddell, M. Am. Soc. C. E.—In discussing this paper I merely wish to point out that Mr. Purdon's curves on Plate II confirm a statement of mine, viz., that, as far as I know, "Class U" of my proposed "Compromise Standard System of Live Loads for Railway Bridges," gives results a trifle greater than those obtained by adopting two of the heaviest engines yet built followed by a train of cars as heavy as are yet used on any track. I have plotted the equivalent uniform loads for "Class U" on Mr. Purdon's Plate II, and find that the resulting curve passes throughout its entire length a little above the curves for the heaviest loads there given. Again, I have plotted also on the same plate the curve for the lightest load of my proposed system, viz., "Class Z," and find that it agrees quite closely

In the following list each railway is given a number which is placed immediately after the title of the road. The wheel weights are expressed in units of 1 000 lbs., and the uniform loads are expressed in pounds per linear foot:

Railway	No.	Loading (wheel spacings and wheel weights, as printed)	Following	Uniform load
W. & L. E. Ry.	1	16 16 — 97 8.1 22 23 — 69 5 7 22 23 — 54* 4 5 22 23 — 54 4.6 22 23 — 85 7 14.5 16 — 68 4.1 14.5 16 — 68 5.6 14 5 16 — 58 4 1 14.5 16 — 108 9 16 15	.... 2d engine	14.5 16 48" 2 5' — 2 240
O. R. I. & P. R. R.	2			
F. Ft. W. & W. RR.	3	12 — 7.6 24 — 4 5 24 — 4 6 24 — 4 6 24 — 10.6 16 — 5 16 — 5 16 — 5 16 — 8 12	2d engine	16 3 — 3 500
C. H. V & T Ry.	4		2d engine	3 000
H. & B. S. RR.	5			
G. & H RR.	6			
E. & W. RR. of A	7			
P Br. Co.	8			
C. & W. M. Ry.	9			
N. & Ft. S. Ry.	10	15 — 97 24 — 69 24 — 54 24 — 54 24 — 85 15 — 68 15 — 68 15 — 58 16 — 108 15	.... 2d engine	15 48" — 3 000
F & P. M. RR.	11			
L. V. RR	12			
T. A. A. & N. M. Ry.	13			
T. & O. C Ry.	14			
W. Ry. of A.	15	14 16 — 8 2 24.64 — 5 8 24 64 — 4.5 24 46 — 4 5 24.64 — 7.6 15.68 — 6 15.68 — 5 15.68 — 5 15.68 — 8 14.56	....	15.68 3
N. Y. O & W. Ry.	16	16 16 — 90 29 — 58 28 — 62 28 — 58 28 — 132 16 — 62 16 — 64 16 — 62 16 — 91 16	2d engine / Train of engines	3 000
K. C. & I A. L.	17	16 — 8 28 — 4 5 28 — 4 5 28 — 4.6 28 — 9 20 — 6 20 — 5 20 — 6 20 — 7 16	.... 2d engine	20 3 — 3 000
M. P. Ry	18			
K. C. O & S. RR	19			
E. & T H. RR.	20	17.6 — 8 28 — 6 28 — 4 5 28 — 4 5 28 — 8 17.6 — 5 17.6 — 5 17.6 — 5 17.6 — 8 17.6	.... 2d engine	17.6 3 — 3 500
E T. V. & G RR.	21	15.6 — 7 4 28 6 — 4 10 28.6 — 4 4 28.6 — 4 10 28.6 — 11 1½ 20 — 4 4 20 — 6 4 20 — 4 4 20 — 10 9 15.6	2d engine	20 — 4 000
D. & R G. RR.	22	20 — 8 30 — 4.6 30 — 4 5 30 — 4 5 30 — 10 16 — 5 16 — 5 16 — 5 16 — 7.5 20	2d engine	16 — 3 000
C. S. Ry.	23	15 — 8 30 — 5.5 30 — 4.5 30 — 4.5 30 — 9 18 — 4 18 — 6 18 — 4 18 — 8	.... 2d engine	18 4 — 3 000
B. C. R. O. & N. Ry.	24			
C. G. W. Ry.	25			
C. H & D. Ry.	26			
C. & M. Ry.	27			
F. C. & P. RR.	28			
O. M. & G. RR.	29			
K. C. Ft. S. & M. RR.	30			
M. C. R. R.	31	16 — 97 30 — 69 30 — 54 30 — 54 30 — 66 18 — 58 18 — 68 18 — 68 18 — 108 16	.... 2d engine	18 48" — 3 000
M. L. S & W Ry.	32			
P. S & L E. RR	33			
P. & R. F. Ry.	34			
St. L. M Br Co.	35			
St. L. S. Ry	36			
T. RR A. of St. L	37			
S C. & P. RR.	38			
S. & N. RR.	39			
C St. P M. & O. Ry.	40	15.5 — 8 30 — 5 30 — 5 30 — 6 30 — 11 20 — 6 20 — 6 20 — 5 20 — 8 15.5	.... 2d engine	20 3' — 3 600
*S. A. L.	41	12 — 8 30 — 5 30 — 5 30 — 6 30 — 7 18 — 6 18 — 6 18 — 5 18 — 9 12	2d engine	18 4' — 4 000
O. C. RR	42	16 — 8 30 — 4 11 30 — 4 11 30 — 4 11 30 — 9 22 — 4 6 22 — 5 22 — 4 6 22 — 8 16	2d engine	22 3' — 4 000
P. L. W. of P.	43	15 — 7 10 30 — 4 8 30 — 4 4 30 — 4 6 30 — 10 9 20 — 4 9 20 — 5 10 20 — 4 9 20 — 8 6 16	2d engine	20 2' 6" — 4 000
G R. & I. RR.	44			
P. RR.	45			
C F. & Y. V. RR	46	16 — 8 30 — 4 6 30 — 4 6 30 — 4 6 30 — 11 6 16 — 5 16 — 5 6 16 — 5 16 — 8 16	.... 2d engine	16 3' — 4 000
G C & C. RR	47			
M T. & M. RR.	48			
*G. C & N. Ry.	49	14 — 8 30 — 5 30 — 5 30 — 5 30 — 11 17 — 5 17 — 5 17 — 5 17 — 8 14	2d engine	17 3' — 4 000
C. & A. RR.	50	14 — 7 30 — 5 30 — 5 30 — 5 30 — 10 17 — 5 17 — 5 17 — 5 17 — 8 14	2d engine	17 3' — 4 000
L S. & M. S. RR.	51	14 — 7 30 — 5 30 — 6 30 — 5 30 — 10 17 — 5 17 — 5 17 — 5 17 — 14	Train of engines	
C & O. RR.	52	17 — 7 8 30 — 4 10 30 — 4 4 30 — 4 10 30 — 11 6½ 20 — 5 20 — 4 10 20 — 5 20 — 9 5 17	2d engine	20 3' 5" — 4 000
N N. & M. V RR.	63			
B R & P. Ry.	54	16 — 7 5 30 — 5 30 — 4 30 — 5 30 — 10 6 20 — 4 8 20 — 5 3 20 — 4 8 20 — 8 5 16	2d engine	20 3' — 4 600
C. RR. & B. Co. of G.	55	16 — 8 30 — 6 30 — 5 30 — 5 30 — 11 18 — 6 18 — 6 18 — 5 18 — 8 16	2d engine	18 3' — 4 000
F. RR.	56	15 — 8 30 — 5 30 — 5 30 — 5 30 — 9 3 20 — 4 20 — 6 20 — 4 20 — 10 9 15	2d engine	20 4' — 4 200
C. C C. & St. L.	57	17 — 8 31 — 4 7 31 — 4.3 31 — 4 7 31 — 10 22 — 5 22 — 5 22 — 5 22 — 8 17	2d engine	22 4 6' — 4 000
D. L. & W Ry.	58	20 — 8 32 — 4 6 32 — 4 6 32 — 4 6 32 — 8 6 22 — 4 6 22 — 6 22 — 4 6 22 — 9 20	2d engine	22 3 6 — 3 600
L & N. Ry	69	7 8 32.5 — 6 8 32.5 — 4 6 32.5 — 4 6 32.5 — 9 6 22.5 — 4 8 22.5 — 7 1 22.5 — 4 8 22.5 — 6	....	2' 6"
N C. & St. L. Ry.	60	20	2d engine	22.5 — 3 500
N. P. RR	61	3 000 — 4 16 — 8 34 — 4 9 34 — 4 6 34 — 4 9 34 — 12 6 20 — 4 6 20 — 5 6 20 — 4 6 20 — 8 16	2d engine	20 4' — 3 000
W. RR.	62	15 — 7 10 35 — 4 8 35 — 4 4 35 — 4 8 35 — 11 2 18 — 5 18 — 5 19.5 — 6 19.5 — 8 4 16	2d engine	19.5 — 3 500
C. & E. I RR	63	15 — 6 35 — 4 6 35 — 4 6 35 — 4 6 35 — 7 23 — 5 23 — 5 6 23 — 5 23 — 8 16	.... 2d engine	23 3' — 4 000
N. Y. L E. & W RR.	64			
N. D. & C. RR.	65			
E. & T. H. RR	20	20 — 8 35 — 6 35 — 4 5 35 — 4.5 35 — 8 20 — 5 20 — 5 20 — 5 20 — 8 20	.... 2d engine	20 3' — 4 000
M C. RR	66	14 — 7 36 — 6 36 — 5 36 — 5 36 — 10 22 — 4 6 22 — 6 6 22 — 4 6 22 — 8 14	2d engine	22 4 — 4 000
P & R. RR.	67	8 1 36 — 4 11 36 — 4 4 36 — 4 6 36 — 12 10 20.5 — 6 20.5 — 6 7 22.5 — 5 22.5 — 7 6	....	22 6 4
L. I. RR.	68	16 16 — 36 — 36 — 40 — 36	2d engine	4 000
C. V. RR.	69	16 — 8 40 — 4½ 40 — 4½ 40 — 4½ 40 — 13 22 — 6 22 — 5½ 22 — 5 22 — 8 16	2d engine	22 4' — 4 000
A. G S. RR	70	16 — 8 40 — 4½ 40 — 4½ 40 — 4½ 40 — 8 22 5 — 4 22.5 — 6 25 — 4 25 — 8 16	.... 2d engine	25 3 — 4 000
L. N. A. & C. Ry.	71	16 — 7 11 40 — 4 10 40 — 4 4 40 — 4 10 40 — 10 8 17 — 6 6 17 — 5 1 20 — 6 5 20 — 7 7½ 16	Train of engines	
L. V. RR	72	16 — 97 40 — 69 40 — 54 40 — 54 40 — 85 18 — 68 18 — 68 20 — 58 20 — 108 16	.... 2d engine	20 48' — 4 000
E. J. & E. Ry.	73			
D. & H. C. Co.	74			
P. & R. F. Ry.	34			
A. T. & S. Fé RR.	75	18 — 7.5 44 — 4.5 44 — 4.5 44 — 4.5 44 — 10.5 20 — 6 20 — 6.5 22 — 5 22 — 8 18	.... 2d engine	22 6' — 3 200
U P. Ry	76	17.3 — 8 9 32.4 — 4.8 37 3 — 4.4 39.9 — 4.8 37.6 — 11.6 17.5 — 6 17.5 — 5.2 17 5 — 5 17.5 — 13 8 17 3	2d engine	17.5 — 3 000
		A uniform load and single concentration, equivalent to above loading to be used.		
W. & C. Ry	77	Two 83 ton consolidation engines		3 000
P. & N. Ry	78	Two 94 ton consolidation engines		4 206
I. Ry. of C.	79	15 — 90 24 — 58 24 — 52 24 — 66 24 — 91 21.25 — 64 21.25 — 81 21.25 — 64 21 25	This loading is used for floor beams and stringers.	
B. of Q. Ry & N. Co.	80			
Q. C. Ry.	81			
P P J. RR.	82	Uniform load of 6 300 lbs. per foot for 25' span, decreasing to 3 200 lbs. for 120' span		
O. & G. V. Ry.	83	Uniform load of 3 000 lbs. per foot and two concentrations of 30 000 lbs. for spans over 120'.		
L. & N. RR.	59	20 — 5 8 20 — 8 6 40 — 9 40 — 7 18 — 4 8 18 — 8 4 18 — 4 8 18		
P. RR	45			
C. F. & Y. V. RR.	46	16 — 5 6 16 — 9 40 — 8 40 — 9 6 16 — 5 16 — 5 6 16 — 5 16		
G. C. & C. RR.	47			
M. T. & M. RR.	48			
C & O. RR	52	20 — 6 6 20 — 8 8 40 — 7 6 40 — 9 8 20 — 5 20 — 4 10 20 — 5 20 — 9 1 20	.... 2d engine	20 3' 6" — 4 000
N. N. & M. V. RR.	53			
D. L. & W. Ry.	58	20 — 5 6 20 — 6 6 42 — 8 6 42 — 8 6 22 — 4 6 22 — 6 22 — 4 6 22 — 9 20	.... 2d engine	22 3' 6" — 3 600
C. V. RR.	69	16 — 6 16 — 8 44 — 7 44 — 10 20 — 5 20 — 5 20 — 6 20		
N. C. & St. L. RR.	60	6 3 19.35 — 8 4 32.7 — 9 33.19 — 7 10 16.66 — 4 4 16.63 — 6 3 18 75 — 4 4 18.75 — 2 3	.... 3 500	
C. C C & St. L. RR.	57	4 000 — 20 — 6 20 — 6.6 20 — 8.6 45 — 7.5 45 — 11 5 22 — 5 22 — 5 22 — 5 22 — 10.5 20	.... 2d engine	22 4.6' — 4 000
N. Y. N. H. & H. RR.	84	18 — 8 40 — 6 6 40 — 7 40 — 8 22 — 5 22 — 6 22 — 5 22 — 8 18	2d engine	22 — 4 000
N N & M. V. RR.	53	16 — 8 25.6 — 4 3 25 6 — 4 3 25 6 — 4 3 26.0 — 4 3 25.6 — 7 6 20 — 4 8 20 — 6 7 20 — 4 8 20 — 7 3 16	.... 2d engine	20 2' 3" — 4 000
C. & O. Ry	52			
R. & D. Ry.	86			
A. C. L.	86			
M. & N. G. Ry.	87	20 — 8 30 — 4 6 30 — 4 6 30 — 4 6 30 — 4 6 30 — 16 6 20 — 5 20 — 5 20 — 5 20 — 10 20	.... 2d engine	20 3' — 3 600
L E. & W RR.	88	5 10 — 4 2 — 7 — 9 — 7 3 18.25 — 4 9 18 25 — 5 3 18.25 — 4 9 18 25 — 8 9 11	.... 2d engine	18.25 4' — 3 000
C. C. C. & St. L. RR	57	4 000 — 4 6 11 15 — 6 6 15 — 4 6 36 — 5.7 38 — 6.1 36 — 11.1 22 — 5 22 — 5 22 — 5 22 — 9 15	.... 2d engine	22 4.5 — 4 000
D. L. & W Ry.	58	18 — 5 6 18 — 4 8 40 — 5 6 40 — 6 40 — 8 6 22 — 4 6 22 — 6 22 — 4 6 22 — 9 18	.... 2d engine	22 3' 6" — 3 600
N. & W. RR.	89	4 000 — 10 34 — 4 8 34 — 4 8 34 — 4 8 34 — 10 .... 4 000		
N. & W. RR.	89	4 000 — 7 6 40 — 5 40 — 7 6 — 4 000		
W. N. Y & P. RR.	90	13.2 — 7 1 27.12 — 4.6 26.68 — 4.4 30.51 — 4.8 29.65 — 9.9 18.724 — 4 1 18.724 — 6 18.724 — 4.1 18.724 — 8.9 13 2 — 2d engine — 18.724 — 7.5 21 76 — 5 21 76 — 19 21.76 — 5 21.76 — 7.7 21.76 — 5' 21.76 — 19'		
A. & St. L. Ry	91	Uniform load of 12 000 lbs per foot for 7' span, decreasing to 1 000 lbs. for spans over 60'		
N. Y. C & H. R RR.	92	The above load preceded by a concentrated load [60 000 + 3 200 (p — 8)] to be used for web members of trusses. Where p is the panel length and is greater than 8.		
N Y P. & B. RR.	93	Uniform load of 4 000 lbs. per foot, and a single concentration of 40 000 lbs.		
C B. & Q. RR.	94	Uniform load of 2 840 lbs. per foot. Additional uniform load of 820 lbs. per foot, 105' long. Additional uniform load of 3 680 lbs. per foot, 15' long.		
D. & I. R RR.	96	Uniform load of 4 200 lbs. per foot, with a single concentrated load of 45 000 lbs. for panels less than 15'; and of 56 000 lbs. for panels between 16' and 20' long.		
N. Y. & N. E. RR.	96	Uniform load of 4 000 lbs per foot. Additional uniform load of 3 000 lbs. per foot, 25' long. Alternative concentrated load of 50 000 lbs.		
B. & M. RR	97	Uniform load of 30 000 per foot for 4' span, decreasing to 4 340 lbs. for 150' span.		
O. & M. RR.	98	Uniform load of $\left\{ \begin{matrix} 3\,500 \text{ or} \\ 3\,000 \end{matrix} \right\} \left\{ 1 - \frac{120 - l}{100} \right\}$ for spans less than 120' long. Uniform load of 3 500 or 3 000 lbs. per foot for spans greater than 120' long.		
J. T. & K. W. Ry.	99	Uniform load of 3 600 to 3 000 lbs. per foot.		
St. L. & S. F. Ry	100	Uniform load of 6 700 lbs. per foot for 14' span, decreasing to 3 000 lbs. for 153' span.		

* The spacing of these loadings is not accurate, but is very nearly correct.

Purdon No. 663.

with the curve for the lightest loading given on the plate, viz., that for the New York Central and Hudson River Railroad.

Now it is probable that Mr. Purdon, in preparing Plate II, chose the heaviest engines he could find for the greatest loads, and the lightest heavy engines used on trunk lines for the smallest loads; hence it seems fair to conclude that my proposed "Compromise Standard System" gives the heaviest and lightest loads that are needed at present in designing bridges. In order to make the system suitable for the Santa Fé specifications, however, I have added a "Class T," to be used for floor systems and primary truss members only, the engines for which correspond approximately with the typical ones adopted in the Santa Fé specifications for 1893.

The diagrams of equivalent loads, etc., for the proposed "Compromise Standard" will be issued soon.

C. D. Purdon, M. Am. Soc. C. E.—My paper was prepared simply to show how a few engines, thought remarkable enough to exhibit, compared with the loads which bridges were supposed to carry, and in this respect was hardly open to discussion. I had no intention of reflecting on standard loading, or starting anew a discussion on that subject.

Mr. Baldwin's table gives very valuable information as to the difference of opinion in bridge loading, and, so far as I know, is the only attempt ever made at a classification of the same.

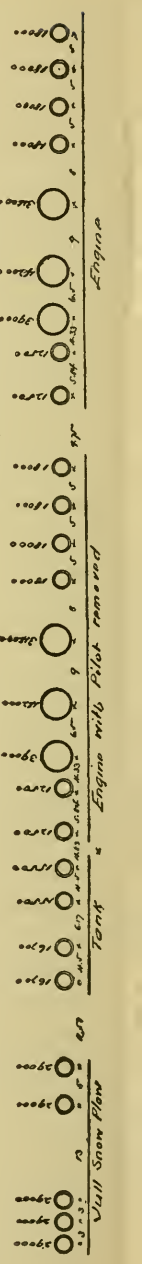
Mr. Baldwin says that a study of specified loads would appear to promise more fruitful results than a study of actual loads. Perhaps so, but certainly the actual loads should also be studied for comparison; it is the actual load which comes on the bridges.

I think diagrams of the engines in Mr. Baldwin's table, prepared as were my diagrams, would show that the uniformity of loading so much advocated by Mr. Waddell really exists, but is not recognized. A diagram thus prepared would give a number of groups of loadings, each group of which might be covered by a single typical loading—for instance, in my Plate I, leaving out the A. T. & S. F. and Wabash, the other loads come practically into three groups, the highest of which is covered by Mr. Waddell's "Class U."

The diagram would enable the engineer of any road to see at once, without spending any time in figuring, into what class of loading his specification came, and would do more towards Mr. Waddell's proposed uniformity than any amount of discussion and figures.

It would be considerable labor to prepare such a diagram, but it could be divided up among several members, were any sufficiently interested in the subject to devote some time to the calculations. I could undertake 5 to 10 loadings myself.

I notice in Mr. Baldwin's table that only 3 specifications out of 85 require any load to precede the engine. Now, with roads using snow plows, this might make an important difference. Mr. Temple, Assoc. M. Am. Soc. C. E., and I are now calculating the effect of a Jull snow plow—which was on exhibition at Chicago—followed by two engines, and find it makes quite serious differences in the strains in some bridges, especially in the floor. The plow has an ordinary tank behind it, and this tank is pushed by two engines, the leading engine having its pilot removed. The following is a diagram of the concentrations:



This loading increases the strain by the following:

On 10-ft. span	.....	24 %
On 15 "	.....	22¾ "
On 20 "	.....	15 "

On 30-ft. to 40-ft. spans, no difference; and on spans from 40 to 90 ft., about 2 or 3 per cent. All these on stringers or plate girders.

Our figures on truss members are not yet complete. So far we find an increase of 6% on a span of 164 ft.

These increases are over the two engines on the diagram, followed by 3 200 lbs. per linear foot.

I do not see why the uniform load following the engines should be varied to suit the engines, unless in case of a mineral road, or one which does not exchange cars with any other road.

Under the present system of traffic there is nothing to prevent a car, loaded in New York, going over every road of the same gauge in the United States and Canada; and, as Mr. Baldwin says, this extreme load is about 2 650 lbs. per foot; so that 3 200 should be a sufficient typical load, unless in the case of a road using special ore cars.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

664.

(Vol. XXX.—December, 1893.

SURVEYS FOR RAILWAY LOCATION.—DISCUSSION ON PAPER No. 619.*

By GEORGE R. HARDY, G. B. NICHOLSON, C. O. GLEIM, ARTHUR PEW, A. BRYSON, W. R. KING and F. A. GELBCKE.

GEORGE R. HARDY, M. Am. Soc. C. E.—The very excellent paper that is now under discussion divides the work into four parts, namely: Reconnaissance, General Survey, Detailed Survey and Field Location.

Personal experience reminds us that closely following the examination of a country, which the author designates as reconnaissance, we are accustomed to survey some trial line which will afford a base for detail examination, until superseded by the location work. For our long railroad lines, and particularly the transcontinental, geographical exploration is necessary before such reconnaissance. The information thus obtained enables a selection of passes or features of the probable routes to be further investigated, these belonging to the class of more stupendous obstructions.

The exploration indicates the places for profitable reconnaissance. The reconnaissance determines which general routes are worthy of a

* "Surveys for Railway Location." By F. A. Gelbcke, Engineer of Government Railways at Cologne, Prussia. *Transactions*, Vol. XXIX, page 429.

trial survey. The trial lines with topography provide data and maps for projecting a location line. Trial lines are usually run by angles and courses, while the location line is staked every 100 ft. in curves and tangents. Engineers of the highest grade and experience are demanded for personal examination of the country, and on their ability and decisions much depends.

Our methods offer quite a contrast with the suggestions under discussion.

First.—As to instruments. For exploration—barometer, sextant, compass and camera. For reconnaissance—barometer, compass and hand level. For trail lines—transit, level, tapes, chains and rods; recently the stadia work has been used. For location—transit and level, etc., etc.

Second.—Our corps of engineers are not so amply provided with technical members, nor are we able to avail ourselves of such well-advanced mapping.

Third.—We are not accustomed to depend so much on topography as to establish the location line, beyond the necessity for careful examination of the proposed location in the field, with freedom to make such changes as are warranted.

G. B. Nicholson, M. Am. Soc. C. E.—Mr. President, I think the gentleman who preceded me has not comprehended the German methods. Probably Mr. Gleim can correct me if I am not right, but I understand that in Germany topographical surveys of the country are so complete and detailed that a railroad line may be located on published maps which would be as perfect as the usual preliminary survey made without the aid of such maps.

C. O. Gleim, Cor. M. Am. Soc. C. E.—You can in some of the provinces; in a great many cases the surveys of the Government have not been completed.

Arthur Pew, M. Am. Soc. C. E.—It is certainly advisable that the country should be thoroughly studied before the line is finally located. The locating engineer is an important factor in making or marring the fortunes of a road, and, in order that he may obtain the best line, he should have all the information that would be of service to him.

The methods proposed by Mr. Gelbcke for obtaining this information are slower and more expensive than those that are customary in this country. The reason for this is, that he does not fully utilize the

skill and judgment of the locating engineer, but has an expensive corps of engineers collecting information, a large proportion of which an expert locating engineer would see from a simple inspection of the country was unnecessary.

It is customary in this country for the locating engineer to make a careful examination of the country through which the line is to run. He has with him, of course, an aneroid barometer, a pocket compass, and sometimes other instruments. The country may be such that he can quickly decide on the best route, or there may be several possible routes, between which he cannot decide without an actual survey. As soon as he decides which line he wishes to run first, a corps is put in the field, and the line run in the usual manner by setting numbered stakes 100 ft. apart.

The work of an ordinary corps is from 1 to 10 miles of line a day, depending on the character of the country, efficiency of the corps, etc. The cost of a corps varies from about $800 to $1 500 per month, with the number of men and salaries paid.

There is rarely any objection to staking out the preliminary lines run in this country, for property owners generally understand, or are informed, that each is only a preliminary line, and that the final location may possibly be several miles away. The fact that several preliminary lines are usually run, and the consequent uncertainty as to which will be followed most closely by the final location, has a tendency to prevent any considerable amount of extortion on the part of landowners. These preliminary lines are plotted on a scale suitable to the character of the country and, being staked out on the ground, serve as base lines by means of which the necessary topography can be taken rapidly and accurately. The locating engineer, with the assistance rendered by these maps, can select the best route and locate the line at a minimum of expense.

This is the usual American method. Its advantage over that proposed by Mr. Gelbcke is that it obviates a great deal of unnecessary work and expense, and in general gives excellent results. So far as my own observation extends, American roads are located fully as well as those of Europe.

In my own experience, in some of the southern and western States the total cost of location, including all preliminaries, has rarely amounted to as much as $150 per mile of located line, even where the

country was thoroughly studied; and, in some instances, the cost has not exceeded half that amount.

A. Bryson, M. Am. Soc. C. E.—Mr. Gelbcke has given us a very interesting paper on surveys for railway location, but the great detail of his method will appear rather extreme to his American *confrères.* One is greatly impressed with its thoroughness, but it is very questionable whether the results of that system, practically carried out, would be any better than one not so elaborate under competent direction.

In the method outlined by the author, the entire subject is more and more developed by a series of examinations and resulting maps, until the selection is finally made by a process of elimination. All this requires time, patience and draughtsmen; also, the entire absence of an American board of directors.

The reconnaissance is unquestionably indispensable, and if the previously mentioned board of directors have not already determined on the general points to be touched, so practically defining the route, the engineer will have an opportunity to show his genius for "general" location in contradistinction to "detail" location; if he possesses the "diagnosing" faculty, his personal preliminary work will save much time, so that the author's "General Survey" and "Detailed Survey" may be brought together into one.

The author recommends "in the case of narrow valleys, always to consider both slopes of the valley in the extent of the survey." If this is intended to mean simply the taking of general notes sufficient to intelligently determine questions of line fitting which may arise, the writer's views are in hearty accord, but not if carried to the extent that was done in one case of which he knows. A railway line was located in a valley varying, maybe, from 200 to 1 500 ft. wide, having bold hills on each side, so that there was no possibility of doing anything except to stick to the flat valley. Nevertheless, cross-lines were run 500 ft. apart, from up on the hill on one side to some equally necessary point on the other, throughout the entire distance of 15 or 20 miles.

There can be no "hard and fast" rule applicable to all occasions. If the engineer engaged on such work is possessed of that personal acumen, like a physician's intuitive ability to diagnose diseases, much time and money will be saved; it is aided by education, study and

practice, but they do not entirely make up for its absence; otherwise, one physician would be as able and successful as another, and, among engineers, there would exist a like equality.

The fact that location may make the difference between dividends and bankruptcy when the road is in operation is difficult to impress upon the average director's mind, but the engineer should insist upon a most careful and thorough personal investigation before he sends a party into the field. For this purpose a hand level and compass, or other handy instrument, in the hands of a man who knows country when he sees it, will settle questions of choice of routes in much less time than exhaustive barometric surveys, as advocated by the author; and when supplemented by some flying levels to critical points will, in the writer's opinion, be more satisfactory, his experience with the aneroid having generated a feeling in his soul somewhat akin to distrust.

The author's advocacy of contours and paper location is most cordially endorsed. Possibly his service, on two separate occasions years ago, under "practical" men impressed upon the writer's mind the absolute necessity of experimental work and the contour map, as well as the great saving in time and work they effect. Each of those men "located as he went along"; if the line happened to "hit about right," "go ahead"; if not, and in rough country it was generally "not," we had to "back up" and try again.

The engineer's judgment and perception are requisite in taking the notes, in making the map, and in laying the line. His science and knowledge will both find occupation, but they do not entirely take the place of that faculty which every successful locating engineer should possess, of selecting and judging line and country, to a degree, without instruments.

Much of the bad location to be seen in the United States is due to interference of directors, local influences, and insufficient time for the preliminary examinations. The writer recalls one case when something more than a month was spent in preliminary work over more than 90 miles of line, part of which was rendered necessary because some one in authority "had an idea" and the engineer's objections were overruled. After the contour map was completed, the location was projected, and work pushed so rapidly in the field, that, notwithstanding the country was very broken, no time was given for revision,

which in one or two places he desired to make, but contractors were at work as soon as the assistants could set the cross-section stakes. This is a common occurrence in this country, however, and is only mentioned to show that the author's system of selecting a route by the slow process of elimination is not generally adapted to our requirements.

A slight additional cost of construction, due to lack of time for all the finer adjustments which extended study might show to be possible, is more than offset by the earlier realization from operation, or at least is so considered by those who control the finances. If the author's words, "the surveys for railroads should therefore not be regarded lightly," could have been impressed upon the minds of American railway directors, we should have fewer instances of bad location.

Colonel W. R. King.*—Forty years ago, Colonel George Escol Sellers, a mechanical engineer, then living in Cincinnati, in connection with a Mr. Whetstone, constructed a kind of velocipede, a two-wheel affair, that would go over the ground and give you a profile, showing horizontal and vertical distances, and do it with a fair degree of accuracy. It could be run at about four miles an hour and give a profile of the country. That instrument was made for the Government, and in some way it got boxed up in Washington, where I found it, a great many years ago, and took it to Willett's Point. It was on exhibition in the Engineer Section of the Government Building at the Columbian Exhibition. I found Colonel Sellers a couple of years ago; he is now living in Chattanooga, Tenn. This idea is re-discovered every little while, but this particular invention was made and tried in 1853, and it is a very creditable instrument; it will pay any one to look it over.

Mr. F. A. Gelbcke.—I have been much gratified by the interesting discussion called forth by my paper, and participated in by prominent engineers, showing that American engineers concur in appreciating the great value of good railway surveys.

In answer to the remark by Mr. Pew, that according to his observations American railroads are located equally well as those of Europe, I beg to say that in Europe, including Germany, the location of railways in former years used to be carried out in a similar manner as in America. It is only within the last twenty years that a change has taken place in Germany, by the frequent adoption of the methods described in my paper.

* Corps of Engineers, U. S. A.

Some twenty years ago I took part in the preliminary work for the St. Gothard Railway, in Switzerland, which gave me an opportunity for recognizing the great value of well-elaborated contour maps. As early as 1862 Mr. Wetli, C. E., had prepared such a map for the St. Gothard Railway, on a scale of 1 in 10 000. This map was successfully used for the great number of studies made, with the object of adjusting the line as well as possible to the difficulties of the case, and as a comprehensive map of the whole territory of the project was referred to over and over again. Nevertheless, the chief engineer, Mr. Gerwig, had contour maps made on a scale of 1 in 2 500 for the further elaboration of the best line; and when the line had finally to be located for construction, the then chief engineer, Mr. Hellwag, did not shun the time and expense of having contour plans made on scales of 1 in 1 000 and 1 in 500. It was only by this means that the design of the St. Gothard Railway could be elaborated with such perfection, as presented to the visitors of the World's Columbian Exposition in some prominent details.

On returning from Switzerland, nearly twenty years ago, and resuming my practice in Germany, I soon found full opportunities for applying the experiences gained in the system of railway location, and my first step was to break with the method previously prevailing in Germany, of prematurely staking out trial lines in the field. The results of the change were highly satisfactory. A large number of younger colleagues, now advanced into prominent positions, after assisting in elaborating and introducing the new method, have gradually propagated the knowledge and application of it over Germany, effecting, as I feel convinced, an important improvement.

The new method advocated does not, however, do away with the requirement of "judgment and perception" in the engineer in charge of the work. The development of the best line of location will leave him full opportunities for displaying these qualities. But he can do this work with much better success, and he can bring proof of the line recommended by him being the most advisable one. In many cases the surface of the country gives so little chance of judging from mere observation, that the best location can only be found by the aid of plans, a fact that only recently I have found confirmed in a most surprising manner.

As to the time and money required for railway surveys according to

the method described, I beg to repeat that they cannot bear comparison with the great advantages and savings effected. And in case there is need of very quick work, that can also be effected under this method by taking regard of the principal conditions of the case, but leaving aside all less important accessories.

If the locating engineer would guard himself against disappointments and waste of money, he should choose for his surveys the way indicated, which is sure to lead to the desired end. That there are simple cases, forming an exception, in which an early staking out of the line in the field may be adopted, has been already remarked towards the end of my paper. But in most cases the early setting out of the line in the field is a mistake, and a thorough elaboration of the project from the general to the detail only to be recommended. I hope that the ideas advocated by me may gain ground with my American colleagues and bear good fruits.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

665.

(Vol. XXX.—December, 1893.)

CALCULATION OF CROSS-SECTIONS.—DISCUSSION ON PAPER No. 620.*

By ARTHUR PEW and FRANCISCO DA SILVA RIBEIRO.

ARTHUR PEW, M. Am. Soc. C. E.—Except in rough or mountainous work, it is rarely the case that a railroad cross-section cannot be measured with sufficient accuracy by simply taking the center height and two side heights.

Whenever these three heights give a correct profile of the surface, no matter what the slope of the ground may be, the cross-section may be reduced to a nearly equivalent level cutting (having the same base and slope ratio) whose height is one-fourth of the sum of the side heights and twice the center height.

The area of this new section will always slightly exceed that of the original section. This is an advantage, as it tends to compensate for the slightly concave surface which the earth, in theory at least, always presents to a railroad excavation or embankment.

Let a and b represent the side heights, and c the center height of a

* "A New Method of Calculating Cross-Sections of Roads and Railroads." By Francisco Da Silva Ribeiro, Director of Public Works, Aveiro, Portugal. *Transactions*, Vol. XXIX, page 447.

cross-section. Reduce this to a level section, making its height

$$= c' = \frac{a + b + 2c}{4}.$$

The area of the level section will exceed that of the original section by $d^2 r$, where $d =$ the difference between c and c', and $r =$ the ratio of the horizontal distance of the side slope to the vertical distance.

If desired, a table can be easily calculated giving the amount of this excess for different values of d, and from this table the proper corrections can be taken ; but on moderately sloping ground, the excess is so small that it may be neglected.

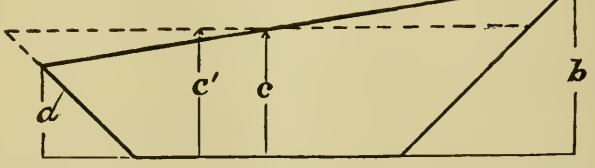
The advantage of reducing sloping cross-sections to equivalent level ones is that the end areas need not be calculated at all. The cubic contents can be taken directly from tables, and corrected for longitudinal slope, as explained in several text books on the subject.

I believe the formula given to be original. By its use the calculation of excavation and embankment becomes a very small affair, as, after a man becomes familiar with it, he can calculate from 500 to 1 000 cross-sections a day of such ground as constitutes at least 95% of railroad work, not in mountainous regions.

This method, however, is only applicable to sections that can be measured by taking the center height and two side heights. On very irregular ground the method of Mr. Ribeiro, or something similar, may be employed.

M. Francisco Da Silva Ribeiro.*—Mr. Arthur Pew, in discussing our method of calculating cross-sections, remarks:

1st. That a simpler system for computing cross-sectional areas in a flat country consists in reducing each section to a nearly equivalent level cutting, whose height, $c,' = \frac{a + b + 2c}{4}$.



2d. That the surface so calculated will always be slightly in excess of the exact area, but the excess, whose value is represented by $d^2 r$, or $\frac{d^2}{t}$, d being the difference $c' - c$ of the two center heights, is so small on

* Translated from the French by Foster Crowell, M. Am. Soc. C. E.

moderately sloping ground that it may be neglected, and for other cases a table can be prepared giving, for different values of d, the correction $\frac{d^2}{t}$, which is to be deducted, to obtain the correct area.

3d. That the method of equivalent level cuttings enables one person to compute easily from 500 to 1 000 cross-sections in a day.

4th. That our method is not advantageous except in rough ground, where the system of three heights cannot well be employed, and that, even there, some other system would give as good results.

The writer thanks Mr. Pew for his able criticism, and will now endeavor to answer him.

The process by the three heights is of course very ingenious and simple, but it requires previous plotting of the sections; whereas, in our method the plotting is not indispensable. In comparing the number of sections which it is possible to compute per day, the time spent in plotting must be taken into account; with our diagram one employee can calculate 200 sections per day, and the writer does not think it would be possible for him to plot and calculate the same number by the three-heights method.

As regards accuracy, furthermore, Mr. Pew's method is not to be compared to ours, for the expression $\frac{d^2}{t}$ varies, not only with the inclination of the surface, but also with the center height; really the exact value of the height of section reduces to—

$$y = \frac{c + l\,t}{\sqrt{1 - \frac{i^2}{t^2}}} - l\,t$$

l being the half width of platform, and i the slope of the surface; the difference—

$$y - c = (c + l\,t)\frac{1 - \sqrt{1 - \frac{i^2}{t^2}}}{\sqrt{1 - \frac{i^2}{t^2}}}$$

$$= \frac{i^2}{t^2} \times \frac{c + l\,t}{1 - \frac{i^2}{t^2} + \sqrt{1 - \frac{i^2}{t^2}}}$$

We see that this difference increases with $\frac{i}{l}$ and with c. Let us suppose a case of a fill on ground inclined at $\frac{1}{3}$; making $t = \frac{2}{3}$ and $i = \frac{1}{3}$ we deduce—

$$y - c = 0.177\ (c + l\,t);$$

if $c + l\,t = 20$, which corresponds to a fill of 17 or 18 m., we find—

$$y - c = 3.54;$$

and it follows that $\frac{(y-c)^2}{t} = \frac{d^2}{l} = 18.80$.

The error must increase in applying the three-heights' method since c' is greater than y. In our method there are no appreciable errors.

The correction table proposed by Mr. Pew is the weak point of his system.

In conclusion, the author knows no other process that gives the cross-sectional areas on any ground without plotting and with such close approximation.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

666.

(Vol. XXX —December, 1893.)

THE GAUGES OF RAILWAY TRACK.—DISCUSSION ON PAPER No. 621.*

By H. G. PROUT, FOSTER CROWELL, E. E. R. TRATMAN and E. A. ZIFFER.

H. G. PROUT, M. Am. Soc. C. E.—I do not rise to discuss this paper at any length, but it ought not to go into the printed records of the Society without a word of protest. Those who have read it will see that the whole argument is a collection of the assertions which we were very familiar with in America twenty years ago, but there is no demonstration. The author says that "economy of construction is principally brought about by the faculty of narrow-gauge roads to adapt themselves closely to the formation of the country and to use steeper gradients and smaller radii," and the economy of construction from the use of lighter materials is particularly dwelt upon. But a 4 ft.-8½ in. gauge railroad can be built as light as a 3-ft. gauge railroad, if you build it for the same character and amount of work. If you want a light secondary road to do a small traffic, with light loads hauled short distances, your rolling stock and your rails and all your structures can be light; but for heavy traffic they must be heavy. The author

* "On the Gauges of Railroad Track in General, with Special Consideration of Narrow-Gauge Railroads." By E. A. Ziffer, M. Am. Soc. C. E., of Vienna, Austria. *Transactions*, Vol. XXIX, page 453.

speaks particularly of the economy of rolling stock. As a matter of fact, a narrow-gauge locomotive will cost more than a standard locomotive per ton of useful weight. Of course, all this is within reasonable limits, but we are speaking of a railroad, built where it is worth while to build a railroad, and not a mill or plantation railroad. The author speaks frequently of the economy of maintenance and operation arising from the narrow gauge, as, for instance, the economy of operating small station buildings, small yards, small everything, but he neglects to mention the very great economy he would derive from having small men to run his locomotives and turn his switches, etc. Finally, his demonstration of the economy in operating expenses is based upon a few railroads cited, in which the ratio of operating expenses to gross earnings is favorable; but he tells us nothing of the character of the traffic or even the rates received for carrying that traffic; and that, of course, is fundamental. Those are special railroads, built and operated under special conditions; it is probable that they pay special rates, and it is possible that, with larger gauge, the ratio of operating expenses to gross earnings would have been still more favorable.

Foster Crowell, M. Am. Soc. C. E.—I quite agree with what Colonel Prout has said. I think he might have said a little more, in calling attention to the practical demonstration of the failure of the narrow gauge in this country, where very few of the many narrow-gauge roads of the last decade are now running as such. There are some, it is true, that are still running, but with a third rail for the standard gauge. A great many narrow-gauge roads have been abandoned, and others, that were built during the craze of ten or fifteen years ago in places where standard gauge might have been, have been remodeled into standard gauge at considerable expense. The reason has been the excessive cost of operation of the narrow gauge, all things considered. The question of the initial cost of construction is not one to be considered by itself. I have recently had occasion in my professional practice to examine a group of minor railroads, some standard and some narrow gauge, all built for the same purpose of obtaining lumber and bark from otherwise inaccessible timber districts, and where the roads are not intended to continue very long in service. Even there the comparison of the working expenses showed that, while the four standard and six narrow gauges had been successful, the operation of the standard-gauge roads

had been much more profitable. Wherever a narrow-gauge road is either in competition or in communication with a standard gauge, it is at a disadvantage; and even in a new country, without competition, the physical argument will generally be in favor of the standard gauge, although, very obviously, conditions are sometimes met with which justify the narrow gauge. The arguments for the narrow gauge, at this time, from our experience, are rather ancient history.

The rise and fall in this country of the narrow-gauge delusion is illustrated in the following little table, which I have prepared to show the comparative mileage at the present time and of ten years ago, when the narrow-gauge movement had just passed its high water-mark. The figures of total mileage given therein are taken from Poor's "Manual of Railroads," editions of 1883 and 1893; the manual does not summarize the narrow-gauge roads, but I have carefully gone over the statements of steam railroads in the two editions and separated the two classes.

It may surprise many to learn from the table that no less than 147 narrow-gauge railroads, with an aggregate length of 9 291 miles, have been either entirely converted to standard, supplied with a third rail, or abandoned during the past eleven years. These roads, "though lost to sight," are, doubtless, "to memory dear" to the confiding shareholders, who were led astray by the same specious arguments that we have had presented anew in this paper.

It is scarcely necessary to add to this record the experience afforded in other countries as to the comparative values of the narrow and the standard gauge, but it may be stated broadly that wherever the two have been introduced into the same territory the standard gauge has proved the best fitted to survive. In the interesting paper presented to this division of the Engineering Congress, upon Railroads in the Republic of Mexico in 1893, Señor E. Prieto Basave, M. Association of Engineers and Architects of Mexico, makes the statement that, while three standard-gauge roads have paid, respectively, approximately 6½, 8½ and 13½ per cent. per annum, the fourth of the group, the Mexican National, which is narrow gauge, has only realized about 4 per cent. It is only fair to say that Señor Basave does not himself attribute the difference in earning power to the gauge, but ascribes it to the road "not being very well managed." He adds, however, that "at the present time it is paying the tribute due to its wasteful construction,

for, notwithstanding its being a narrow-gauge road, it has cost 50% more per kilometer than the Central" (standard gauge):

Railroads in the United States (excluding street railroads).	Miles.	Number.
All railroads in operation in 1882*	112 412	1 538
Narrow gauge in operation in 1882*	9 762	151
Percentage, narrow gauge in 1882	$8\frac{7}{10}$	$9\frac{8}{10}$
Narrow-gauge railroads built during the decade ending with 1892	4 281	102
All railroads built during same period	62 811	2 166
Average narrow gauge per year	428	
Total average per year	6 281	
Percentage, narrow gauge	$6\frac{8}{10}$	$4\frac{7}{10}$
Narrow-gauge railroads built in the three years, 1890, 1891 and 1892	278	12
All railroads built during same period	13 870	
Average narrow gauge per year	93	
Average all roads per year	4 623	
Percentage, narrow gauge	2	$4\frac{3}{10}$
Narrow-gauge railroads either converted to standard 4 ft. $8\frac{1}{2}$ gauge, supplied with third rail, or abandoned, during the decade ending with 1892	9 239	143
Narrow-gauge railroads in operation in 1892 †	4 804	110
All railroads in operation in 1892 †	175 223	2 166
Percentage, narrow gauge	$2\frac{7}{10}$	5
Narrow-gauge railroads converted to standard gauge subsequent to 1892	52	4
Narrow-gauge railroads, included above, used exclusively for light passenger excursion business	190	12
Narrow-gauge railroads doing combined passenger and freight business in 1893	4 562	94
Percentage, narrow gauge compared with all roads	$2\frac{6}{10}$	$4\frac{3}{10}$

* Poor's "Manual of Railroads," 1883.
† Poor's "Manual of Railroads," 1893.

Returning to the United States railroads, it should be noted that the comparisons in the table are based upon the length of road, and not of total track; but as few narrow-gauge roads have second track, and the total length of main track, excluding sidings, in the United States in 1893 could not have been far from 200 000 miles, the percentage of narrow gauge for this year, as given in the table can properly be reduced in the proportion of 200 000 to 175 000 to $2\frac{3}{10}\%$ of the total mileage.

The table shows also, that for the past three years the proportion of narrow gauge built has been very far below the average for the past ten years, and has been confined to short lines averaging 23 miles in length; these figures are probably not absolutely exact, but they have been carefully collated and undoubtedly fairly reflect the passing of the narrow gauge.

E. E. R. Tratman, Assoc. M. Am. Soc. C. E.—In this country the necessity and advantages of uniformity of gauge have been proved beyond dispute.

The Billerica and Bedford Railway, instanced by Mr. Ziffer, was changed from 2 ft. to 4 ft. 8½ ins. gauge some years ago.

In ordinary railway service there is little advantage *per se* in the narrow gauge, and the advantages claimed can be secured with standard gauge if proper attention is paid to location, construction and equipment.

It has been amply proved in this country, Australia and India that it is unwise to introduce a second gauge.

Mr. E. A. Ziffer.—I am indebted to the American Society of Civil Engineers for having afforded me an opportunity, as I was unable to be present at the Chicago Congress, to reply to the adverse criticism raised in the discussion of my paper.

First of all, I may, I think, conclude that Messrs. H. G. Prout and Foster Crowell were laboring under the erroneous impression that I advocated the adoption of the narrow gauge for railroads generally, and that the justification of the advantages was matter of recent date.

This is not so, inasmuch as I showed that in Germany the movement in favor of narrow-gauge roads commenced in 1869, and that in 1870, before the British Railway Union in Liverpool, Fairlie declared that "every inch added to the absolutely necessary width of the gauge increases the cost of construction, the proportion of dead load and the cost of passenger traffic." Nor did I fail to quote the Festiniog narrow gauge which dates from 1832.

My task was to set forth the value of the narrow gauge for lines of subordinate importance, secondary and vicinal railroads, feeders, etc., and to describe the developments and extensions of recent years, especially on the Continent of Europe, for I am well aware that the narrow gauge is systematically opposed on every side, and frequently by technical men of repute.

I deem it one of the most important aims of engineering to cultivate the construction of secondary railways, which, in consequence of low cost, are yet dividend earning with a minor traffic, and are on the other hand suitable to deal with traffic of gradual growth, or with traffic developed by extraordinary causes.

In my final conclusions I stated distinctly that for secondary (local

and vicinal) railroads, as well as for all roads of inferior importance, which subserve limited interests and accommodate principally local traffic, standard and a great variety of narrow gauges have been adopted. The latter have been chosen in just appreciation of their economic advantages in construction and operation. I also added that in certain cases the adoption of the standard gauge might prove desirable for secondary railroads; but that, as a general rule, the narrow gauge was the rational solution of the question of the simple and economical construction and operation of secondary lines, even for a not inconsiderable traffic, and that the narrow gauge was also to be recommended for main lines in countries with a presumably moderate traffic, and where the financial status necessitated the greatest possible economy, so that the invested capital might earn a good interest as quickly as possible.

If the whole argument of my paper is deemed by the gentlemen I named to be a collection of assertions, and to demonstrate nothing, I must state in reply that their protests are not calculated to throw light on the subject.

It is true that a standard-gauge line can be constructed with light superstructure and light rolling stock; but even in this case the cost of constructing and stocking will be higher, because more land must be acquired and a wider roadbed built, with a greater development in its establishment. The superstructure will cost more in consequence of the longer sleepers and increased width of crown of ballast, and of the increased width of the rolling stock, without attaining any compensating advantage, while in the event of a connection with any other standard-gauge line, the transfer of the rolling stock from the main road is impracticable in consequence of its greater weight.

My opponents' contention may be right, that a ton of narrow-gauge locomotive costs more than a ton of standard-gauge locomotive; but it is undoubtedly true that a narrow-gauge locomotive costs less than a standard-gauge locomotive, and, which is in favor of the narrow gauge, that a certain number of locomotives are needed to work a given number of trains. For example, a locomotive of the Saxon State narrow-gauge line (0.75 m. gauge) costs 21 376 marks, as against a standard-gauge locomotive of the Saxon State railways, which costs 44 902 marks.

Maintenance of the narrow-gauge locomotive..... 1 784.87 marks.
Maintenance of the standard-gauge locomotive ..... 2 695.24 "

In a serious question of the nature of this one, I treat as an ill-placed jest the remark that small men should be found to operate such light lines.

In point of fact, secondary railroads, on which the speed will depend upon the importance and extent of the traffic, can, without diminishing safety, be operated more simply and with a less numerous and experienced staff of employees. This alone constitutes an economy, without any references to the further advantages set forth in my paper. If proof be necessary in respect of saving in cost of construction, this may be obtained easily by consulting the surveys and estimates made for one and the same line on different gauges, to which I also referred, and thereby demonstrated that the cost of construction and width of gauge are inseparably connected. Nor have I in view, as Mr. Prout may think, merely the construction of a mill or plantation railroad; but public lines intended for passenger and goods traffic. I quoted, giving some particulars, several lines operating in various countries, which it is inferred that I selected on account of the favorable ratio of their working expenses to income, without giving any particulars of their traffic. Had I desired to give these particulars, I should have been forced to compile statistics, which would have exceeded the limits of a paper, and this would have been superfluous, for having quoted the sources from which my information was drawn, every one could procure the desired details for himself.

Mr. Foster Crowell's statement, that the narrow gauge has been demonstrated to be a practical failure in America, that some of the roads have been remodeled into standard-gauge railroads, and that others have been provided with a third rail for the standard gauge; further, that operating the standard-gauge roads had been much more profitable than operating the narrow gauge, and that the arguments for the narrow gauge at this time are ancient history, all go to show that either the narrow-gauge roads have been constructed in unsuitable cases or in ignorance of the conditions, and not in accordance with modern tested practice, in narrow-gauge construction, and, perhaps, that they have been operated as though they were standard-gauge main lines.

If all the defects of narrow-gauge roads were so uncontested, it were hardly conceivable that in America, during the last ten years, new narrow-gauge roads should have been constructed, though their average length be only twenty-three miles.

The reference also to Mr. C. Prieto Basave's report merely goes to show that in Mexico the narrow-gauge roads have been badly constructed, and not very well operated.

The communications in my paper are based on practical results, and are in no way the vague assertions which Messrs. Prout and Foster Crowell assume them to be. The results appeared to me, indeed, to be suited to throw light upon an important question. It may be gathered from the paper that narrow-gauge roads continue to extend in European countries in consequence of their higher economic advantages in point of construction and operation.

The governments of Prussia, Russia, France, Belgium and Austria have regulated the application of the narrow gauge by certain enactments, of which those in Prussia and Russia, issued as late as 1892, are worthy of special attention.

The resolutions of the International Railway Congress and of the International Permanent Tramway Union, which resolutions, therefore, may be deemed to be those of competent bodies, bear witness to the economic importance of the narrow gauge for secondary railroads.

The following resolutions, which could not be embodied in my paper, were passed in September last by the Congress of the International Permanent Tramway Union, held at Buda-Pesth:

"In consequence of the saving which can be effected in their construction and operation, narrow-gauge railroads are destined to form an economical and valuable adjunct in completing the existing network of railroads, and to influence the prosperous development of railroads of inferior order.

"Each of the three gauges in use up to the present time, 1 m., 0.75 m. and 0.60 m., has been found useful in practice under given conditions, and their application, therefore, is justified. The choice of gauge should, however, in each case follow the careful consideration of the conditions presented by detailed investigations of the existing and future probable requirements of traffic, due regard being given to local needs, the convenience of passengers, and the financial and other support available.

"In order to secure the earning of suitable profit on the capital invested in such lines, the limited aims of which are to be realized at low cost, due regard must be given to economical construction and simple cheap operation."

In conclusion, I should like to state that in Bosnia and Herzegovina a continuous network of narrow-gauge railroads, with 0.76 m.

gauge, and 609 km. in length, is in operation, which meets heavy military requirements, although the average cost of construction was 53 206 florins* per kilometer, in despite of the exceptionally heavy character of the country, and the rendering permanent a road laid temporarily in the first instance.

During the year 1891 the traffic of the Bosnia line per kilometer averaged 73 271 passengers and 65 411 tons, and the average gross receipts per kilometer, 4 350 florins. The working expenses amounted to 1 893 florins, and the capital earned a dividend of 4.6 per cent.

The Salzkammergut local railroads, 64 km. in length, and the Styrian local provincial railroads, 49 km. in length, were constructed in consequence of the satisfactory results of the Bosnian and Herzegovinian lines, with a corresponding gauge of 0.76 m. Having been in operation for one year only, no returns are yet available. This year 76 km. of local railroads have been under construction in Styria on the same gauge, and within the last few days the local railroad Ibsthal, 70 km. in length, with a gauge of 0.76 m., has been chartered.

In the Kingdom of Saxony the first narrow-gauge local railroad, with a 0.75 m. gauge, was built in 1881. At the present time there is a network of 17 narrow-gauge lines, 282.15 km. in length, equal to 10.54% of the whole of the Saxon railroads. The average cost of construction per kilometer of the standard gauge amounted to 289 498 marks; of the narrow-gauge lines, to 73 120 marks.

These facts alone, drawn from my native country, and within my own experience, ought to form an irrefragable argument that the employment of the narrow gauges, for secondary and local railways, as well as for extensive connections, with not unimportant traffic is both justifiable, suitable and profitable.

In any case the opinions expressed at the Congress by the two technical gentlemen in question will not in any way influence the movement in favor of the narrow gauge in Europe where the cry is becoming ever louder for cheap local railroads.

* Austrian currency.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

667.

(Vol. XXX.—December, 1893.)

RAILWAY SIGNALING.—DISCUSSION ON PAPER No. 622.*

By GEORGE R. HARDY.

GEORGE R. HARDY, M. Am. Soc. C. E.—It is worth noticing that this is a very exhaustive paper, and is so extended as to preclude a detailed discussion of its many points. The author delineates the various kinds of signals and the methods of using them in different countries. He mentions those in use here, but seems to have overlooked the Sykes system of block signaling.

We are interested to note the tendency to separate the considerations of the means used to actuate the signals from the engineering of operation. Invention has made splendid progress with the first part, but the scientific analysis of the second is still defective. This may be due to the natural conservatism which is encountered when engineers presume to offer to executive officers suggestions as to the business of operation. To a large degree the introduction of signaling becomes revolutionary.

* "Railway Signaling." By G. Kecker, Betriebs-Director, Imperial Railways of Metz, Germany. *Transactions*, Vol XXIX, p. 491.

We will not now attempt to describe or classify the many variant conceptions of the use of signals. Signaling is an element of the operating services, and in a twofold capacity may protect an obstructed track, or expedite the speed by its guarantee of a clear track.

We have seen in this country, contemporaneously with the development of signaling, the rapid strides of improvement of railway plant and devices. Signaling struggled slowly when we were banging our cars over the old stub switches, but advanced more rapidly when we changed to the high rail and planed points. Rearrangement of yards and terminals came with signal towers, enabling the same length of tracks to answer for increased number of train movements.

The author devotes considerable space to remarks on the subject of the colors used in signaling. Speaking of the German practice, we quote: "By this arrangement the locomotive engineer never passes a red signal." Again he notes the experiments of the Chappe Brothers, which showed the visibility of three kinds of lights, viz., white, red and green, was as 1, $\frac{1}{3}$, $\frac{1}{5}$. Again he says: "Movable glass blades or screens, especially if exposed to the weather, are easily broken, and should this occur it might happen * * * that an accident might result. * * * It seems undesirable to arrange signals in such a way that a red or green light might by accident be changed to white."

We therefore find the author has given three good reasons for some modification of the usual plan of colors for signals:

First.—The circumstance of an engine passing a red signal.

Second.—The existing inversion of the adaptability of the lights as regards their visibility.

Third.—The possible accidental changing of a colored signal to white, meaning all clear, when the reverse is the case.

It was mainly due to a careful consideration of the first reason that the system of signals used on the Boston and Albany Railroad was arranged. With regard to visibility of lights, it is safer to extend the warning of danger with the more perspicuous and use the one of lesser visibility for the all-clear signal; therefore, the green light has been tried on some railroads, and found best as an all-clear signal. Should the green screen or lens be accidently removed, the white light so uncovered would not be likely to cause an accident.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

668.

(Vol. XXX.—December, 1893.)

TRANSMISSION OF POWER IN OPERATING CABLE RAILWAYS.—DISCUSSION ON PAPER No. 623.*

By A. BRYSON, DANIEL BONTECOU and ROBERT GILLHAM.

A. BRYSON, M. Am. Soc. C. E.—Mr. Gillham is to be congratulated on having given to the Society such an able and exhaustive paper upon a subject of such great engineering interest and financial importance. Had a description of the alignment in each case, however, been added, or the total number of degrees of curvature been placed in another column of the tables, with the ratios of diameters of curve pulleys and their journals, clearer analyses, and, possibly, general formulas applicable to nearly all cases might have been deduced.

It seems to the writer that exception may properly be taken to the "Average Horse-Power per Train," in Table No. 1, as being somewhat misleading, in that it is deduced from the net horse-power only, after deducting that necessary for operating all the machinery. This is not quite fair when comparing results with other systems of power-transmission for transportation purposes. All the power generated is used somewhere and must be paid for, and, as the work actually performed,

* "Transmission of Power in Operating Cable Railways." By Robert Gillham, M. Am. Soc. C. E. *Transactions*, Vol. XXIX, page 543.

with the entire cost of doing it, is the result which it is desirable to know, each unit of work must be charged, not only with the actual net power expended, but also with its proportion of the power consumed in working up to it. Were this view of the case taken in making up the last column in Table No. 1, the amounts would be increased about $2\frac{3}{4}$ times.

In the tests of carrying pulleys, which are most excellent and instructive, the result of comparing Nos. 2, 4, 5, Table No. 2, is that the horse-power consumed per 1 000 ft. of rope at 7.3 miles an hour varies as the weights of pulleys and as the ratios of their diameters.

When the Walker rings are used it would be desirable to know how much more tension is needed on the cable for a given service than with the solid drums. If the difference is great it might become a serious question on a line having long cables, considerable curvature, heavy grades, and heavy traffic, notwithstanding they overcome some of the very serious objections to solid drums where no provision is made for slip.

As to the increased power necessary for increased speeds. An investigation of Cables 1, 2 and 4, of Table No. 6—they being nearly of the same length and with little or no curvature—shows a coefficient of .03, and that the horse-power per linear foot of cable increases very nearly as V^2.

This, however, does not agree with the tests mentioned as having been made at the Fifty-fifth Street Power Station of the Chicago City Railway, where "it was found that, in order to drive the empty cables at 13.76 miles per hour, 300.6 H. P. was developed, while to drive them at 9.65 miles per hour only required 184 H. P."

In this case if the increment were as V^2, the horse-power for the higher speed, assuming that at the lower speed to be correct, would be 374.1 H. P. The aggregate length of these cables, however, is 59 700 ft.; whereas, in the other tests given in Table No. 6, the longest of the three considered is 24 300 ft. Can these tests properly be compared, and, if so, is this more favorable showing of the longer line due to its better condition and absence of curvature, or is there a factor of the increment proportional in some way to length and weight of cable and the driving machinery, due to and varying as V^2 atmospheric resistance of wheels, drums, etc.

From the Fifty-fifth Street Station the maximum tension on cables during operating hours, when travel was at the maximum, is given as

10 420 lbs., and the maximum load is in Table No. 10 stated to be 167 tons; hence, to pull one ton at the given speed, 13.76 miles an hour, 62.4 lbs. was applied. In the next case, from the Twenty-first Street Power Station, the tension was 11 425 lbs. and maximum load 924 tons, showing only 12.35 lbs. needed to pull one ton. Such marked difference may be due to various causes; but as no explanation is given, it is thought to be of sufficient importance to justify a request for further enlightenment.

It would also be interesting to study the practical results as given in the tests of power absorbed on curves, but as only the cable speed is stated, and no mention is made of the other necessary data, maximum tension on cable, and diameter of pulleys and journals, no calculations can be made.

It is to be hoped that Mr. Gillham will be able to supplement his valuable paper with the data suggested prior to its publication in the Society's *Transactions*.

Daniel Bontecou, M. Am. Soc. C. E.—There have been so few precise records made of the details of cable railway power transmission, that Mr. Gillham's tables form a valuable addition to the stock of current information.

It is not practicable, with certainty, to analyze the resistances of machinery and cables in great detail, but the friction of all the machinery with the cables off the drums may be assumed without material error, at 10% of the total average power required, or, say, 5 H. P. per unit of rope, and the power required by the unloaded cables can be determined with reasonable accuracy.

Table No. 5 of the paper seems to me somewhat misleading in purporting to show that under any probable conditions the brake action of the Cleveland friction clutches could be so much. It is evident from the conditions stated that the total of 802 H. P. includes the engine and line shaft friction 5 times and the friction of one clutch 12 times. While on the other hand it is difficult to understand why, if the total machinery and rope friction was actually 476 H. P., the total power with 64 cars on the line could have been as low as 509 H. P., plus resistance of rope drums.

A test made of the new plant of the Washington and Georgetown Company showed that the friction of three Weston-Capen clutches on the driving pinions amounted to 29 H. P.

The figures given in the tables for resistances of cables are rather higher than should be realized with good details and maintenance, with the exception of the main line of the Kansas City Cable Company which has 22-in. pulleys and a 7-mile speed; and it is interesting to note that the horse-power for the ropes of that company seems to vary about inversely as the diameter of the pulleys.

The figures given for the rope resistances of the Chicago City line, about 34 H. P. per mile of single and probably straight track, are especially excessive, the large friction in this case being possibly due to heavy and badly balanced pulleys, and in some degree to the speed.

In designing the Washington plants it was assumed that the resistances due to ropes on 14-in. pulleys, and at a speed of nine miles per hour, would be 7 H. P. per mile of straight single track (0.00132 H. P. per foot of rope) and 1 H. P. for 14° of single-track deflection. These figures proved to be 2½% under the truth as developed by indicator tests, and included friction due to all deflections of the ropes, other than those at the curves. These tests, in the case of the new plant which drives three ropes, were made by first indicating the total friction of machinery and cables, then indicating the friction with the various ropes thrown off the drums, but without throwing off the clutches, a method which probably gives more precise results than any other.

Mr. Gillham's figures seem to confirm the fact that the resistances of ropes increase with higher speeds, but in a slightly higher proportion than that of the speeds.

A table which I have prepared gives reliable details of cable railway work which may be of interest in adding to those already presented in the paper (see page 552).

The power per car as given by averages deduced from indicator cards is usually misleading in failing to throw any light on the maximum requirements.

This average for a standard four-wheel car and average load is about 2⅔ H. P.; but even in the latitude of Washington a resistance of at least 8 H. P. per car must be anticipated and in some way provided for, and, when questions of heavy grade are involved, it should be remembered, for instance, that a loaded car on a 10% grade calls for about 25 H. P. In this connection I would suggest that the determination

TABLE

Line.	Engine.	Drive.	Speed. Miles per hour.	Machine friction. Horse-power.	Rope friction. Horse-power.	Rope. Length.	Single track. Deflection Degrees.	Diameter pulleys.	Remarks.
Kansas City.									
Grand Ave. Ry. Co.—									
Walnut St. Line	32" × 48"	Indirect C. I. gear.	8.0	45	223	All ropes.			Pulleys with c. i. spindles 1 in. diameter in lignum-vitæ bearings.
Fifteenth St Line		Direct mort. gear.	9.25	51	305	74 700′	1 071	12″	
Westport Line			10.25	15	40				
Holmes St. Ry	24" × 48"		12.85	18.75	50	18 900′	.....	14″	⅞-in. steel spindles—babbetted bearings.
Washington, D. C.									
W. & G. Ry. Co.—									
Seventh St. Line	20″ and 36″ × 48″ c.	Direct mort. gear.	9	40	1	33 000′	40	14″	⅞-in. steel spindles—babbetted bearings.
Navy Yard Line	36″ × 72″		9	73	104	31 556′	682	14″	⅞-in. steel spindles—babbetted bearings.
Navy Yard Auxiliary		Direct	6		19.5	3 949′	374	14″	⅞-in. steel spindles—babbetted bearings.
Georgetown Line		Rope-drive	9		80	23 588′	910	14″	⅞-in. steel spindles—babbetted bearings.
Fourteenth St. Line			9		95.5	27 817′	680	14″	⅞-in. steel spindles—babbetted bearings.

Note.—Machinery friction given for engine and main shaft only, except in case of 36″ × 72″ W. and G. engine, when it includes all machinery, with ropes off drums.

of the size of engines for a cable road demands very careful judgment in order to insure enough power when needed and to avoid running with an under load during a large portion of the time.

Recent practice seems to have settled on driving the drums directly from the main shaft and with 2-in. rope slings, although a number of plants have been installed on the Pacific Coast driven by a continuous rope. The freedom from noise, vibration, breakages due to sudden strains and back-lash, all of which have proved serious objections to gears, are very decided advantages on the side of the rope-drives, and may be considered as essential ones in a city power-house.

I think Mr. Gillham rests under a misconception in regard to priority in curve construction, and, in fairness to the group of ingenious men who worked out the details of the first San Francisco roads, will state that, while the Kansas City Cable Company's line was not operated until 1885, the Sutter Street Road, which was converted into a cable road in 1876, had two right-angle horizontal curves in which the main rope was run on 20-in. pulleys with details very similar in principle to those in common use now. And before 1884 the MacAllister Street main rope was operated around four horizontal curves of about 45° each. It would, however, probably be correct to say that the Kansas City case was the first application of Miller's duplicate rope system to a curved line.

I notice that Mr. Gillham states that the rings of the Walker drums of the Cleveland plant allow equalization of stress in the cable, and would be glad to know if we are to understand that that fact has been determined with precision in any way.

I have used the Walker drums, and believe, as do many others, that they probably have many advantages over a solid cast-iron drum. I know that the rings move relatively to each other under the conditions of actual work, and that I could not find that there was any appreciable wear on the rings of the Seventh Street plant at Washington after a year's use; but I have never been satisfied that the rings were so sensitive, and accommodated themselves to the strain so regularly as to equalize the rope strain to an appreciable degree. Of course this is purely an empirical matter, and any precise information would be of interest. If the friction on the seat of the rings under the usual conditions of lubrication is enough less than the friction of the rope in the grooves of the ring, to invariably cause the ring and not the

rope to slip under small variations of power, then the equalization of strain would occur and would be evidenced by absence of wear in the groove. This is the only kind of determination which occurs to me as practicable.

Better information about the relation to the resistance of the number of wraps on the drums, and also about the size of drums and the limit of operation with one drum only driven, is much to be desired.

A very interesting paper on "Cable Railway Propulsion," read by Mr. W. W. Hanscom before the Technical Society of the Pacific Coast, in 1884, analyzed the results of operation of the first seven San Francisco roads, and deduced the conclusion that their maximum possible efficiency with cars 1 000 ft. apart was 50 per cent. A comparison of the best of these lines with the best lines of recent construction indicates that the possible efficiency may now be placed at a higher percentage.

Robert Gillham, M. Am. Soc. C. E.—I regret that I am compelled to discuss or reply to discussions of the paper, "Transmission of Power in Operating Cable Railways," here in New York, for the reason that all the data relating to the subject are in my Kansas City office. I will, however, add a few explanations to what has already been written regarding the subject. I fully appreciate the fact that many additional items could have been incorporated in the paper that would have added much to its general interest and value; but, at the same time, for many reasons, it was desirable to limit the scope of the paper.

With reference to the suggestions made by Mr. Bryson, I may say that the degrees of curvature, diameter of curve pulleys and their journals, without complete details illustrating the design of curves, curve pulleys and their journals, would not be of as great value as might at first appear. The tables would certainly appear more complete with this information added, but there would still remain the necessity for explanations.

In a few cases on curves, the curve pulleys are removed and the cable conducted around large 12-ft. sheaves in vaults beneath the tracks. This method is resorted to for the purpose of prolonging the life of the cable, and also to reduce cable friction, and is introduced usually on level curves, also in curves having descending grades in direction of street-car traffic, where the ordinary street traffic will admit.

The cars pass the curves in the first instance by momentum, and in the second by gravity. Two cases of single-gravity curves may be found on the Washington Street line, Kansas City, and one double-track momentum curve on the West Denver line, Denver, Colo. In the last case here given, cars release the cable on entry to curve, and pass around the same by the acquired momentum of cars. The curve pulleys, as has been stated, are entirely removed, and the cable conducted around two large 12-ft. sheaves placed below the street, close to the intersection of tangents of each track.

The Denver curve pulleys were designed interchangeable, and are in all respects alike.

TABLE A.

CURVE PULLEYS.

NAME OF ROAD.	Diameter of pulleys.	Weight of pulley.	Diameter of journal.	REMARKS.
	Inches.	Pounds.	Inches.	
Denver City Cable Railway...	15	108	$1\frac{15}{16}$	Diameters are given from bottom of groove.
Cleveland City Cable Railway.	39	337	$2\frac{1}{4}$	
Kansas City Cable Railway...	34	415	$1\frac{15}{16}$	Main Line.
	34	415	$1\frac{15}{16}$	Independence Avenue.
	23	240	$1\frac{15}{16}$	Summit Street.
	34	415	$1\frac{15}{16}$	East 9th Street.

The average horse-power given in Table No. 1 relates entirely to the power required to move trains above and in addition to that required for machinery and cable friction, and was arranged in this form in order to separate the power-absorbing elements in cable operation. In the use of Walker rings on driving drums, it does not appear that very much additional, if any, tension weight is necessary, compared with solid driving drums, conditions being equal.

The horse-power per linear foot of cable as given in Table No. 6 for Cables Nos. 1, 2 and 4 increases, it is true, very nearly as V^2, but this fact should not be made an absolute basis of comparison with power tests for cable friction on other lines, for the reason that local line conditions vary greatly and are very rarely alike save in the matter of cable length. Cable No. 1, Table No. 6, has a slight curve, the degrees I cannot now recall; but this curve requires six curve pulleys. Again, on one portion of this line, the route passes through an unusual depression, making it almost necessary to use depression pulleys.

Cable No. 2, Table No. 6, has also a slight curve, requiring four curve pulleys, and in addition operates the auxiliary machinery, which in turn drives the Water Street loop.

Cable No. 4, Table No. 6, has no curves, but is required to pass from power station to front vault, around two sheaves; thence to Kirtland Street, around two sheaves; thence through a blind conduit, about ½ mile, to the Payne Avenue line, where it again passes around two vault sheaves. In this case we have the cable passing half around six extra sheaves not found necessary in the other cables.

Thus the statement in remarks of Table No. 6 to the effect that there were "practically no curves" is a little misleading when the data given is used in comparisons similar to that made by Mr. Bryson. The conditions associated with the operation of each of the above cables vary greatly.

The tests in Table No. 6 cannot be properly compared with the test at the Fifty-fifth Street power station, Chicago City Railway, for the reason that the conditions are widely different. The cables tested at Fifty-fifth Street are straight, having no curves or depressions of any kind, and only two vault sheaves each, except the Hyde Park loop on Fifty-fifth Street rope.

With regard to the difference in power required per ton of load as developed in the Chicago tests, I have not the data at hand to make further statements on this question.

Mr. Bontecou, in discussing the paper, seems unable to understand why the brake action in the Cleveland clutch test should have been so great. The subject-matter on page 556 (Vol. XXIX) of the paper describes the causes of this excessive brake action, but perhaps the statement is not quite as clear as it should have been. I will, therefore, make a further explanation of this test, that it may appear clearer and be better understood. Mr. Bontecou's test of brake action of Weston-Capen clutches in Washington agrees quite closely with similar tests made by the writer with clutches of this design that were in ordinarily good adjustment and repair, but the point in the Cleveland case is one that embraces the question of design in important details, which details are radically different from those involved in the Washington, Denver or Kansas City clutches.

The Cleveland clutches were designed by the manufacturer, and placed on the machinery as Weston-Capen clutches. The fact is, they

fully covered the principle upon which this clutch is designed, except in the method of forcing the friction plates together. Instead of using the usual toggle principle, for the purpose of throwing the circular steel plates, in the sleeve of the clutch pinion, in contact with the circular steel plates, attached to and which rotate with the main shaft, the wedge principle was used, differing very materially from the well-known toggle. The use of a movable wedge for the purpose of forcing the clutch plates together resulted, in this case, in reducing the horizontal movement of the movable plates more than one-half. In the clutch in question, the horizontal movement was entirely too slight, and, with the lost motion in the clutch lever and connections, the clutch plates were more frequently left in contact with each other than out of contact, even though the clutch lever was thrown entirely out.

On more than one occasion when the clutch lever was thrown out entirely, and the plates should have been out of contact with each other, they remained in contact sufficiently to continue propelling the cable. It was this condition of affairs that was determined in the test indicated by Table No. 5 in the original paper, and which fully accounts for the brake action of 292.47 H. P.

The details in the clutches were afterwards changed, additional horizontal movement of plates secured, resulting in minimum clutch friction, and the correction of the difficulty noted.

The results given in the paper, relating to and indicating the power required to propel cables, may, as Mr. Bontecou suggests, seem in some cases a little high, which is not chargeable to defective designs in road machinery, but may, to a great extent, be due to varying conditions and indifferent maintenance of road machinery. None of the railways where tests were made were on dress parade.

Mr. Bontecou has submitted very valuable and interesting data relating to other cable railways, for which it is claimed most excellent results are secured, and this leads him to remark, with reference to the power required to overcome cable friction on the railways reported in the original paper, that the cable friction "is rather higher than should be realized with good details and maintenance."

Comparing the data presented by Mr. Bontecou from cable railways with which he, as designing engineer, was associated, with those referred to in the original paper, possesses interest.

TABLE B.

ILLUSTRATING THE AVERAGE HORSE-POWER REQUIRED PER LINEAR FOOT TO OVERCOME CABLE FRICTION, THE AVERAGE SPEED OF CABLES, AND THE TOTAL LENGTHS OF CABLES, IN THE RAILWAYS IN THE VARIOUS CITIES IN WHICH TESTS HAVE BEEN MADE (NOT INCLUDING MACHINE FRICTION).

LOCATION OF RAILWAYS.	Average rope friction, horse-power per linear foot.	Average speed, miles per hour.	Total cable length in feet on lines tested.	REMARKS.
Kansas City, Mo.— Walnut Street Line....(B.) Fifteenth Street Line....(B.) Westport Street Line....(B.)	.0070	9.16	74 700	This is the highest cable friction reported. For degrees of curvature, see table by Mr. Bontecou
All lines, Grand Avenue System, Kansas City....(B.)	.0066	10.08	93 600	
All lines, Washington, D. C.(B.)	.0028	8.5	119 910	
Seventh Street, Washington D. C....(B.)	.00124	9.0	33 000	Seventh Street cable is most favorably located, and has no right-angle curves, and is practically straight.
All lines, Kansas City Cable Railway....(G.)	.0031	8.7	94 132	
Main Line, Kansas City Cable Railway....(G.)	.0017	7.3	22 000	Two double right-angle curves
Washington Street Line, Kansas City Cable Railway...(G.)	.0029	12.0	23 759	One double and one single right-angle curve; one single gravity curve.
All lines, Cleveland, O.....(G.)	.0031	10.8	105 400	Five double right-angle curves.
West Payne Avenue, Cleveland, O....(G.)	.0023	12.0	24 050	No curves.
All lines, Denver, Colo.....(G.)	.0036	10.0	155 617	Five double right-angle; two, 45°; one, 30°; one, 20°; one, 25°. One double momentum curve.
All lines, Chicago, Ill.....(G.)	.0059	12.1	112 920	

NOTE.—The lines marked with "B" are from Mr. Bontecou's table, and those marked "G" from Mr. Gillham's tables. The speed for Washington Street cable was noted in original tables as 8 miles per hour; it should have been 12 miles.

I am very pleased to accept the correction made by Mr. Bontecou, with reference to the date and place where main cables were first operated around right-angle curves. The San Francisco engineers have not received the full share of credit due them, and I regret that this error occurred in the paper.

I should have stated that the Walker rings "tend" to equalize the stress on the rope in passing around the driving drums, as I did not intend to make the broad statement that the rings "equalize the stress on the rope." The fact that the rings usually change their relative positions to each other indicates the absence of rope slip, and the absence of wear in grooves of rings tends to indicate equalization of stress.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

669.

(Vol. XXX.—December, 1893.)

THE CONTINUOUS SUPERSTRUCTURE OF THE MEMPHIS BRIDGE.—DISCUSSION ON PAPER No. 624.*

By GEORGE H. THOMSON, HENRY GOLDMARK, Sir BENJAMIN BAKER, ROBERT MOORE, CHARLES MACDONALD, GEORGE S. MORISON and ALFRED D. OTTEWELL.

GEORGE H. THOMSON, M. Am. Soc. C. E.—The author calls attention to a matter which is often overlooked in competitive design, that of "rigidity" of structure. There is in the railroad truss bridge specification generally a lack of provision for rigidity, probably on account of the lack of a full and complete formulation of a theory regarding rigidity of structure, or, as I term it, "structural motion." I regret very much that Mr. Morison has not laid before us his ideas regarding "structural motion." It would seem that at present our competitive designs for railroad bridges, constructed by the usual specifications, are very defective; there is no regard for "structural motion" whatever. In recent designs for railroad bridges I generally design with a provision for "structural motion." While truss bridges for railroads

*"The Continuous Superstructure of the Memphis Bridge." By George S. Morison, M. Am. Soc. C. E. *Transactions*, Vol. XXIX, page 573.

erected in the State of New York must fulfill certain requirements of the railroad commission, so that all parts of structures must be within certain limits laid down by that commission, there is no objection to making some of these parts heavier to provide for "structural motion." I would like to have Mr. Morison say something about this subject.

Wm. Metcalf, Pres. Am. Soc. C. E.—Are there any further remarks? It is hoped that members will at least ask questions and start up some kind of a discussion of these papers which the authors have taken so much trouble to present.

Mr. Thomson.—Mr. Morison is very willing, but he wants some of us to lead him out. He could first define what is meant by "structural motion," or, as he might term it, "vibration," a word which has a different meaning in different countries, but taking the ordinary term, "rigidity," as used by engineers, I think if he would give us five minutes on that subject we would all appreciate it. He says here, "it is possible to design a structure in which no metal under any ordinary supposition will be overstrained, and yet without such overstrain vibrations can exist which would be utterly inadmissible." I think some of us could design a structure that would meet current requirements, but that would be so full of "vibration" that it would be merely a question of a decade or two as to its endurance. I know a bridge of 150-ft. span that has been visibly overstrained in one year, and another of 125-ft. span that did not withstand it so long. The use of steel with punched holes may have something to do with the cause of failure of these bridges, but the bridges were too full of motion. It would seem to me that if these structures have much of motion, it does not make much difference whether the iron is sustaining 6 000 or 8 000 to 10 000 lbs. per square inch, they will not endure; but if the structure has "rigidity," a mere matter of 15 000 to 20 000 lbs. per square inch will not make so much difference; it may endure for years.

Henry Goldmark, M. Am. Soc. C. E.—Having witnessed the tests to which the Memphis Bridge was subjected at its formal opening, I am glad to bear witness to the remarkable rigidity manifested by this great structure.

Considering the fact that two of the spans are of cantilever construction, the deflections under the heavy test train were very moderate, indicating excellent workmanship, with very little play in the connections of the different parts.

There was also only a slight vibration in any of the separate members, even when the train moved over the bridge at a speed of 30 miles per hour. Under these conditions, there can be no doubt that the material in each member is utilized to the best advantage, and for considerable lengths of span the American system of pin connection fully maintains its position.

Having personally examined, in the field, during the past 10 years, many hundreds of railroad bridges, I have had ample proof that for bridges of more moderate span our ordinary methods of construction have, in the past, been very faulty. The light pin bridges, universal a decade ago, with suspended floor systems and pin-jointed lateral and sway rods, have not proved equal to the demands made upon them.

It is true that train-loads have been much increased, but even under light loads I have found the wear of the loose-jointed parts to be very excessive. In the floor system, riveted attachments have, of late, become almost universal, and it is only a question of a few years before the connections of the truss members will be made by means of rivets in railroad bridges under 175 ft. in length. For lengths of 100 to 115 ft., this practice is even now quite general, and I believe with excellent results.

I have built several spans of 120 ft., with riveted joints, which do not deflect over $\frac{1}{2}$ in. under 50-ton engines, and are almost free from vibration in any member. I am quite sure that this is a better result than is usually obtained in through pin-connected spans.

While the calculation of the strains in lattice trusses is apparently less correct and certain than where pins are used, there can, I think, be little doubt that, for short, and, consequently, light, bridges, the "strain sheet" does not, by any means, fully indicate the work which the metal has to do.

Where experience points so clearly as it does in this matter, it is very important to revise our views on this exceedingly practical subject.

Sir Benjamin Baker, Vice-President M. I. C. E.—Engineers understand that the strain sheet is not always a guide as to the effect of the vibratory strains on different parts of the bridge. I have had an opportunity of studying that in some detail during the last year, as I was called upon to rebuild a great many wrought-iron bridges which had been in existence about 40 years, and in pulling down the old bridges it was possible to see where the work really suffered from undue

stress. I found appearances of greatest wear where nobody would have anticipated if guided by the strain sheet alone. Thus the attachment of the trough flooring to the webs of the main girders is a part that is liable to suffer. In the calculations it is usually assumed that the trough flooring simply rests on the bottom of the flange. It is found best, however, to provide for what you may call the contrary flexure at the connection of the floor and girder and make a liberal allowance for it in rivet area. I have seen web plates entirely cut through with the continual wriggling at that point. The members of a bridge are often subject to secondary stresses reaching the limit of elasticity, and wear and tear occurs, although the calculated strains would be much lower than would account for the facts observed.

The loosening of rivets, also, in certain places is another defect observed in old bridges, and I found that to avoid the same, different attachments and riveting to those indicated by calculations are advisable in many cases.

Then there is a very important point which of course is habitually ignored in all calculations, that is, the initial strains which you get in bridge members, according to the form of cross-section. You straighten cold entire plates and bars, and in doing that I have found, by cutting the parts to pieces and seeing the changes which follow relief of internal stress, that in many cases you get varying stresses throughout the member quite up to the elastic limit, and I believe that in many cases so-called mysterious fracture arises from this, because, while you may nominally have 10 000 lbs. per square inch, you have really got stresses running from perhaps 20 000 to 25 000 lbs. per square inch during the passage of a train on portions of the cross-section.

I have in some cases seen evidences of a fracture beginning in the middle thickness of the metal quite out of sight, and constituting, therefore, a perfectly inexplicable failure, as bit by bit, perhaps a thousandth of an inch at a time, the fracture would grow until it extended through the whole section of the bar.

Robert Moore, M. Am. Soc. C. E.—There is one thing in the design of the Memphis Bridge which appears to me to be of very special merit; that is, the design of the rollers which are placed under the span. This is one of the earliest, if not the earliest, structures in this country in which rollers of that design have been adopted. It is a design which ought to be very much more generally adopted than it

has been. It is probably within the experience of many members who have had occasion to observe the action of existing structures, of old structures, in which corrosion has begun and in which the accumulation of dirt about the rollers has taken place, that the small rollers usually placed under bridge spans do not roll. They become fast and bring upon the masonry and the structure itself stresses which are very great. I have recently had occasion to notice a case of this kind where the refusal of the rollers under two heavy spans has initiated a separation of the stones of the coping. I think that engineers generally will do well to take notice of this fact and provide for it very much as Mr. Morison has done by using sections of cylinders of larger diameter.

CHARLES MACDONALD, M. Am. Soc. C. E.—There is a question involved in this bridge which is of especial interest to the manufacturer, but which should also receive the attentive consideration of designers of long-span bridges. I refer to the difficulty of handling the cumbersome masses which are required for completing individual members or sections of the trusses. In this regard the Memphis Bridge would appear to have reached, if it has not exceeded, the limit of span in which the purely pin-connected method of construction is available. Take, for example, a section of bottom chord at the end panels. This is designed to resist tension and compression. It is composed of two vertical members connected by horizontal latticing above and below. Now, no matter how carefully the work of assembling and fitting may be done in the shops, there comes a difficulty, in transporting the completed member to the bridge site and erecting it in place, which is inseparable from the design.

In turning over such heavy masses distortion is almost certain to result. Pin holes which have been bored perfectly fair and at right angles to the vertical sides will be found out of line when the time comes to drive the erecting pins, owing to the impracticability of fixing such heavy masses by lattice plates.

Other instances of a similar character might be given, but this will suffice to call attention to the subject, an attentive consideration of which may lead to the conclusion that a combination of the European practice of riveting with the pin-connected method familiar in the United States will be found to be the true line of development for long spans.

GEORGE S. MORISON, M. Am. Soc. C. E.—The principal feature which has been brought out in this discussion has been the question of rigidity. There are two sources of motion in a bridge under strain, one perfectly legitimate, and one which should be avoided in every possible way. The legitimate source of motion is the change of length of metal due to strain. The illegitimate source is the lost motion which occurs in details, and I believe that more harm is done to bridges by vibration due to this lost motion than comes from every other source. Two or three years ago a bridge across the Mississippi River at Burlington, which would now be condemned absolutely for its details, but which was fairly considered a good design at the time it was made, was taken down. It had cast-iron top chords and post feet, but the only place where abnormal wear was shown was on the pins near the ends where the counters, which theoretically should not have existed, took hold of the pins; there the wear was very great. It was with a view to avoid lost motion in details that some of the heavy details of the Memphis Bridge, described in this paper, were introduced, and I believe that any one who has been on that bridge during the passage of a train will recognize that it is about as stiff cantilever construction as can be found. Of course it is not as stiff as a very much larger structure could be made, but any one who will examine it will recognize its rigidity as compared with other bridges of the same length of span and general dimensions.

There is one point which was suggested by Sir Benjamin Baker when he referred to strains existing in metal in positions where it could not be seen. Some years ago I condemned a large portion of the pins of a bridge for piping, and it occurred to me that the conditions that produced piping in some pins produced a strain nearly as bad as piping in others. Since that time I have adopted the plan of boring all pins through the center. There is no real difficulty in doing it, and it has at times shown a condition of the metal in the interior of of those pins which made me glad that I had required it. It is valuable, both as a means of detecting flaws which could not otherwise be found and as removing the condition of internal strain which has produced those flaws and which may even produce fracture; it is precisely analogous to the approved practice of boring steamship shafts. In a bridge which I am now building, the entire set of pins forged at one shop, and which were forged under too light a hammer, were rejected

for piping; it was necessary to have them forged at another shop which used a heavier hammer; the new pins showed no defects.

ALFRED D. OTTEWELL, M. Am. Soc. C. E.—In examining the details of this bridge one cannot fail to be impressed with the marked improvement in bridge building which has taken place in this country within the past ten or twenty years.

Ten years ago the European engineer visiting this country could justly criticise the frailty of American bridges, the vibration, the tremor, the rattle of the rods, etc. In the present example, however, the most fastidious critic would be pleased, for the design shows a liberal use of material, and that the efficiency of the details claimed the most attention, rather than economy in material or labor.

Looking at the general plans of this bridge, the question arises, how much, if at all, did this general arrangement of spans and piers differ from the strictly economical arrangement of spans and piers? As this is probably a hard question to answer, I will put it another way. If Mr. Morison had been at liberty to select his own length of spans, could he not have saved considerable money by using shorter and fixed spans for the bridge? My own experience—an experience very much more limited than that of Mr. Morison—makes it a matter of surprise to me that the cost of substructure in this bridge exceeded the cost of superstructure, and raises the question: was there anything in the nature of the bed of the river that made sinking the piers more costly than would be at first supposed?

There seems to be a lack of unanimity among engineers as to what is the most economical length of suspended span compared with the full span. In the present example the suspended span is $\frac{451.66}{790.42} = .57$ of the full span; in the Forth Bridge it is $\frac{350}{1710} = .205$; in Mr. MacLeod's bridge over the Kentucky River, L. S. R. R., it is $\frac{260}{551} = .47$; in Mr. Buck's Verrugas viaduct, $\frac{105}{260} = .405$; in the Poughkeepsie Bridge, $\frac{212}{546} = .39$; in the Sukkur Bridge, $\frac{200}{820} = .244$; in the Red Rock Bridge, $\frac{330}{660} = .5$; in Mr. C. C. Schneider's Frazer River Bridge, $\frac{105}{315} = .33$; Niagara Falls Cantilever Bridge, $\frac{120}{495} = .242$; in the Kentucky and Indiana Railroad Bridge, $\frac{160}{480} = .33$; in the proposed Channel Bridge, $\frac{125}{500} = .25$. Short

or long suspended spans seem to depend upon the unimportance or importance of deflection in the bridge. According to this principle a highway bridge may have a shorter suspended span than a railroad span of the same length; and the above ratio should be greater for short spans than for long spans for both railway and highway bridges.

It is interesting to note that the depths of spans in this bridge show a marked tendency towards the European practice of shallow depths.

The use of bulb angles in lateral struts is a decided improvement over the I form, whether the latter is rolled or built up of ordinary angle irons. The bulb angles at present on the market are too heavy for use in most bridges for this purpose. It appears to me that if the rolling mills would roll a series of smaller sections of this shape they would find a ready market.

Mr. Morison's explanation of the method used in ascertaining the sizes of pins is very satisfactory as far as it goes. It would be interesting, however, to know exactly how the size of a particular pin was determined. The customary process is both long, tedious, and unsatisfactory when completed, so that if Mr. Morison has a shorter and simpler method, the profession, I am sure, will be glad to know it.

The advocates for single intersection must be disappointed in the fact, that this important bridge was built on the double intersection plan. I have for some time thought we shall in long spans revert to the single intersection form of truss employed by Mr. Albert Fink for the 400-ft. channel span of the Louisville bridge. This bridge, it will be remembered, has a Warren primary truss of long panels, each subdivided into four shorter panels, to carry the floor and stiffen the upper chord. This form of truss has the advantages attributable to long panels, definite strains, duplication of parts, concentration of strains, fewer members. Whether these advantages are counteracted by disadvantages remains to be seen.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

670.

(Vol. XXX.—December, 1893.)

BRIDGE SUBSTRUCTURE AND FOUNDATIONS IN NOVA SCOTIA.—DISCUSSION ON PAPER No. 625.*

By K. SUTHERLAND, F. W. W. DOANE, W. R. BUTLER, E. GILPIN, P. S. ARCHIBALD, H. S. POOLE and MARTIN MURPHY.

K. SUTHERLAND.—Dr. Murphy is to be congratulated on the success that has attended his constructions in concrete in Nova Scotia, and I think this is due largely to the interest he has taken, as well as the time he has given in personal supervision, even to the most minute details of the several structures he had in hand.

The widely divergent uses to which concrete can be applied with satisfactory results obtained in practice on some of the railways here, and the interesting papers on that subject read by Dr. Murphy before the Canadian Society at Montreal, has invested the details of bridge substructure and foundations with an interest second only to its importance.

I notice that the author has instanced the concreting of two bridges on the line of the Windsor and Annapolis Railway, namely, the Avon

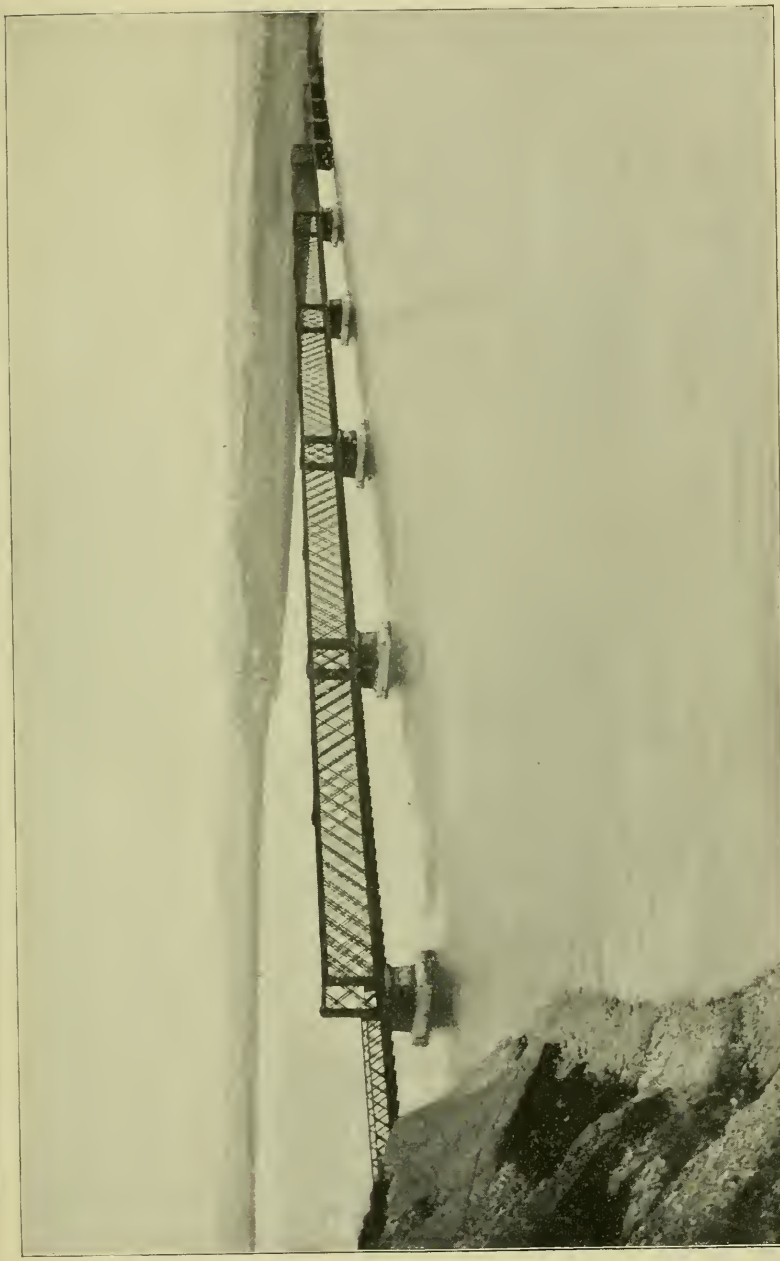
* "Bridge Substructure and Foundations in Nova Scotia." By Martin Murphy, M. Can. Soc. C. E. *Transactions*, Vol. XXIX, page 620.

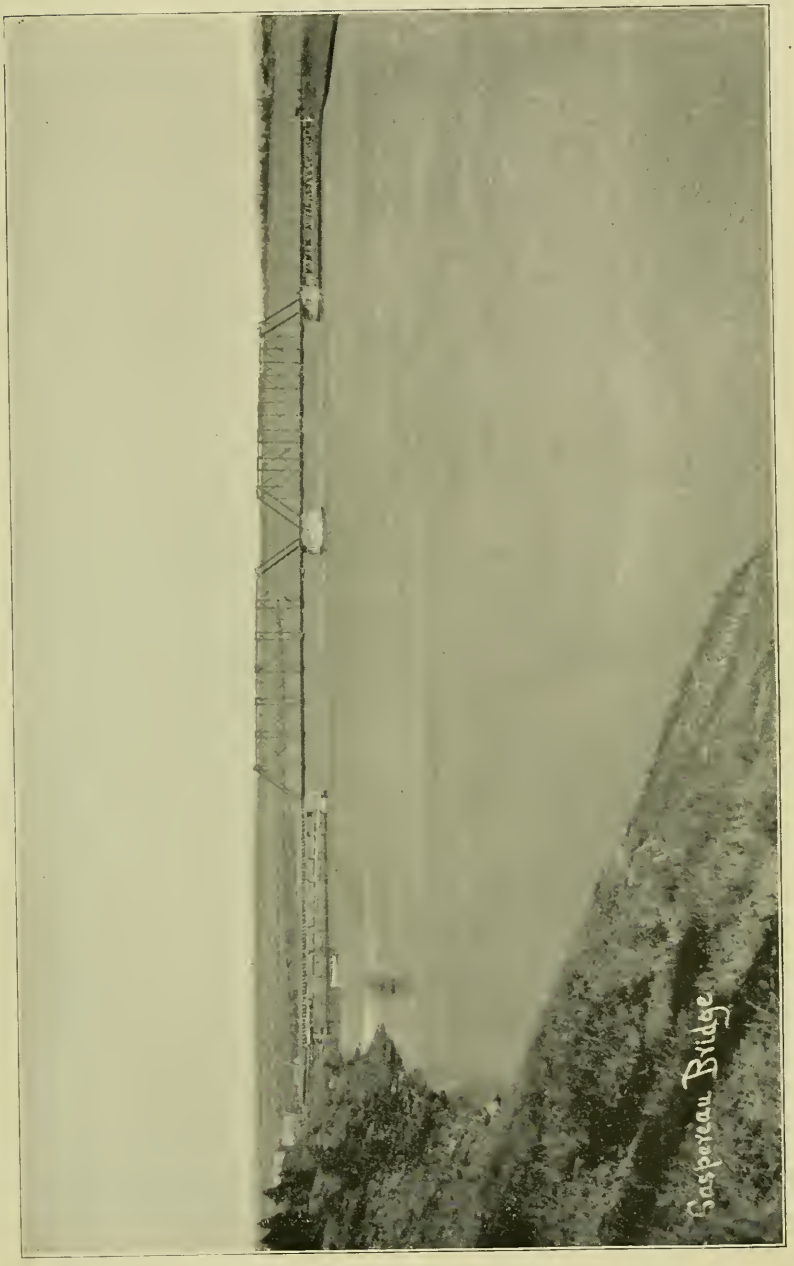
River Bridge, over the Avon, and the Gaspereaux River Bridge, over the Gaspereaux.

Avon River Bridge.—Of the eight piers in this structure, five were concreted by commencing on the outside of the ashlar masonry and incasing them with a matrix to 3 ft. above high water, the outside of which was protected by cribbing of 10 x 10 spruce timber, for the purpose of holding the matrix in place while being deposited.

The estuary of the Avon at this point having a tidal rise of about 28 ft., and a current of six miles an hour, the work of depositing the foundations was attended with some difficulty in keeping the material from being washed out at every influx and ebb of the tide. This was, however, overcome by the knowledge we had gained in putting in our foundations, so that we were enabled in a very few days to put a belt of concrete around the piers of from 1 to 1½ ft. every 24 hours. The proportion of material forming the whole body of concrete was 1 bbl. of White's Portland cement to 1½ bbls. of clean sand, 1 bbl. of clean gravel, and 4 of common field stone. These stones weighed from 8 to 20 lbs., and were all dropped or placed by hand, so that as far as practicable no one stone would touch its fellow. The matrix was delivered *in situ* from top of pier, falling 10 to 30 ft., so that very little, if any, ramming was necessary. The thickness of belt of concrete at foundation was 2½ ft., finishing at 1 ft. 4 ins. This work has now been on probation for over four years, and has so far completely filled the object for which it was erected, namely, the prevention of further disintegration of the bridge piers. This belt to-day clings as closely to the ashlar as on the last day of its completion, without a "threadrent" or displacement to be noticed; in fact, a monolith. The cost was $7 25 per cubic yard. A view of this bridge is given on Plate I.

Gaspereaux Bridge.—This structure consisted of three piers only, and the object of supplying concrete for rebuilding was widely different from that of the Avon River, but the success which had attended the repairs to the latter warranted us in applying the same system, so far as the material of which it was composed was concerned, in the projected rebuilding of the Gaspereaux. The three piers were of the ordinary so-called trestle bents resting on piles capped and interbraced with each other, these piles standing 12 ft. above the surface at low water, with probably 10 ft. below the surface, into the hard river mud. This interbraced piling had become crippled and bent in toward the




Gaspereau Bridge

middle of the river, so that the time had come for reconstruction, either with ashlar masonry or with concrete, as above noted.

The superstructure consisted of two American pin-connected truss iron girders; lower chords of eye-bars; each span being 145 ft. A view of this bridge is given on Plate II.

To build the piers of ashlar would have necessitated the construction of solid false works, to enable us to keep our train service intact, so that the idea of concrete piers, which would obviate the necessity of any false works, and, therefore, the conclusion to build concrete piers was at once, I believe, a judicious and happy movement in the direction of economy as well as facility of construction. The result has justified this. We commenced operations by excavating about 8 ft. below the surface, 6 ft. under low water, until we found hard clay with an admixture of gravel, cutting out only about 6 ft. of plan section at one time, placing therein 3 ft. of field stones, grouting the same with 1 to 1½ of cement and sand, until the whole foundation was completed. We then proceeded with the same proportions of concrete material as in the case of the Avon River Bridge, cutting out nearly half of the timber in bents and filling as the work progressed; completing the piers on top at 8 ft., excepting in the case of the center pier, at 10 ft., which held the ends of both trusses. The tops of all the piers were capped with large and well-dressed hard free stone, chamfered around the edges, and giving the whole a finished appearance. During construction the concrete was kept in place by a wooden framework of 6 x 3 spruce, on the inside of which 1½-in. plank was fastened. The Gaspereaux, although not so wide as the Avon, has 4 ft. more tide, and the same speed in its current. We had no difficulty in maintaining all our concrete in position as deposited, no doubt owing to our having passed the experimental stage at the Avon River Bridge, in placing concrete in the tidal waters of the bay of Fundy.

Mr. F. W. W. Doane.—In discussing Dr. Murphy's interesting and valuable paper, I can testify to the satisfactory results that have attended the use of concrete in Nova Scotia, as I am familiar with most of the concrete work done in the Province.

It is rather surprising that engineers are so conservative in their practice, and that the adoption of concrete as a building material is not more general, since the discovery of the process is by no means recent. One urgent reason for its adoption in the city of Halifax

is the difficulty in connection with the employment of skilled labor.

In work such as Dr. Murphy describes monolithic massiveness is of the first importance, as the resistance to blows from drift ice, logs and rafts, and the power to withstand the freezing and thawing of a Nova Scotia winter, cannot be obtained in quarried stone. The experience so far has not shown any failure from the action of frost on concrete structures in our climate, which is more severe than in the Upper Provinces, in consequence of the rapid and frequent changes in the temperature.

The draw pier of the "Victoria Bridge" referred to by Dr. Murphy has been an unquestionable success. This work was done where there is a very rapid current, and consequently the blows from drift-ice, logs, etc., are very heavy, and the destructive effects of freezing and thawing would be more noticeable in consequence of the great rise and fall of the tides. The drift-ice and other floating material strike as impotently against the pier as if it were a lump of lead, and the freezing has no effect upon it whatever. Compared with the conduct of masonry under similar circumstances, concrete is far cheaper, even if the first cost were more than that of masonry, instead of less.

It is of course important that great care should be used in the selection of material and the method of compounding, as the quality of concrete depends on the material used and the skill in mixing. It is also important that concrete work should be done before cold weather, so that the process of induration shall be sufficiently advanced to prevent the surface from cleaving and disintegrating. Uniformity in the quality of cement is also important.

The construction of concrete work was at first unpopular in this Province. It was thought expensive and was opposed by the public, although it was not exactly an experiment. It was undertaken hesitatingly at first; but it is now employed in the construction of all classes of work.

The building stone in Nova Scotia is generally too far away to compete successfully with concrete in cost. In Halifax, for the reason already given, namely, "the difficulty in controlling skilled labor," it was found to be much more economical and satisfactory to build sewers, sidewalks and catchpits of concrete instead of brick. The sewers are constructed of concrete blocks molded to the proper form

and allowed to set for some time before being used. They are laid by ordinary workmen, who are paid less than half the wages demanded by skilled workmen, who must be employed to construct brick sewers.

Catchpits built of concrete are perfectly water-tight and easily cleaned, and the cost is much less than those built of brick, or the recent invention of cast iron. They are provided with a concrete cover, or, in places where the traffic is very heavy, a granite curb is set in the face and a cast-iron cover used.

W. R. Butler, Assoc. M. Inst. C. E.—Doctor Murphy's paper will command the careful and interested attention of members of the Congress as exhibiting methods bold and novel in character, very successfully adapted to meet cases of somewhat peculiar difficulty. That the several problems presented have been solved by economical methods, but at no sacrifice of permanence or stability, both as to the general designs and also as to the execution of their details, the writer believes will be generally admitted.

Concrete has in many instances proved its superiority to masonry construction in the waters of our tidal rivers, especially where such repeated, rapid and extreme fluctuations of level obtain as in the tidal waters of the bay of Fundy.

The railway bridge across the Avon at this place (Windsor), referred to by Dr. Murphy is a notable example of the deteriorating effect of such influences upon heavy masonry, with which the writer is very familiar. At one hour of the day the external face may be subjected to a water pressure due to a head of from 30 to 50 ft., and a few hours later the same face may be exposed to atmospheric influence; at half tide the rate of rise and fall is approximately 1 ft. in from five to seven minutes; the effect of this, in a temperature far below the freezing point, together with blows from masses of floating ice of 50 tons upwards, may be imagined, the current running, as it often does, some 5 or 6 miles an hour. These masonry piers, in spite of constant watching and repairs, have proved utterly unable to withstand the deteriorating influences, and the treatment referred to by Dr. Murphy has been resorted to.

The design of the highway bridge discussed in the paper and represented in some of the details in Plate I, seems to fulfill all the requirements of the locality—the writer has seen its stability very severely tested. On one occasion by the thrust of a compact body of heavy

ice, filling the estuary below for some miles from bank to bank, and steadily advancing up stream, impelled by wind and tide, each pair of cylinders was cutting its lane through the ice floe, which was more or less cemented together by half frozen slurry from a heavy snow storm, to the accompaniment of much groaning and crushing of ice-cakes and vibration of bracings. The bridge, however, has stood this, and many like tests, unimpaired. As to floating masses of heavy ice or such like objects, and their damaging effects upon a bridge pier, the writer is tempted to advance the observation that it will be less upon a pier presenting its full breadth as a face at right angles to the direction of the current, than upon a similar pier built with a cut-water; at any rate, when the current is a swift one, the floating body is diverted from its course further up stream, the velocity of the current being very sensibly diminished within a curved envelope extending forward from the face of the pier and within which a sort of water cushion is formed. Masses of ice are frequently given a rotating movement and possibly strike a less damaging blow, or are frequently carried entirely clear of the structure.

The deposition of concrete *in situ* in paper bags, as described by the author of the paper, is certainly an improvement upon deposition simply from a "skip" and deposition in bags made from a material which is not immediately destroyed.

It would be interesting if Dr. Murphy would state whether he has had opportunity of examining, after any lapse of time, the concrete deposited in the construction of any of the works described, or has been able to judge whether chemical action of sea-water has taken, or is likely to take, place.

The design of the pivot piers at Lockport and at Bear River suggests the question, "has the water free access to the voids within the structure during tidal fluctuations? If not, is there any danger of forced percolation, and consequent chemical action upon the concrete?"

Doctor Murphy has had long and varied experience in the use of Portland cement concrete. It would be interesting if he would express his views as to what are satisfactory tests of a cement suitable for use in such works as are described, in the proportions specified for rubble and for fine concrete.

(1) As to chemical composition; (2) specific gravity; (3) fineness; (4) behavior under the testing machine. Also, his views as to

the right proportions of water to be used in mixing for deposition under such conditions, and any information as to what are safe and what are unsafe atmospheric temperatures for carrying on concrete work.

On some of these points there seems to be diversity of opinion; ignorance of, or inattention to, some of them not infrequently leads in all probability to the making of a very poor concrete from very good materials.

Dr. E. Gilpin.*—As Dr. Murphy has observed, there are few countries more trying to masonry than Nova Scotia. The constant alternations from thawing to freezing during the winter, often taking place twice during the 24 hours, are very destructive to all structures, especially when they are at all porous. The experience of 80 years has shown that no variety of "sandstone" met in Nova Scotia can be relied upon for permanent structure, and the ravages of time are often serious within a period of 30 years. Two methods of attack characterize the disappointments met in brick and stone structures, the disintegration of the mortar as well as of the stone. The only stones that appear indestructible with us are granite and syenite, and the more highly metamorphosed of the slates of the gold-measure rocks. The latter, locally known as "ironstone," have, in some cases, shown not the slightest weathering for about one hundred years. Limestone has been used to a limited extent. Some very old buildings have stood well; other structures have not yet served their apprenticeship to time, and in the case of limestone each quarry must make its own record.

The concrete age which is beginning in Nova Scotia bids fair to displace stone in nearly every respect where utility is more consulted than appearance. The first application of it in Nova Scotia for colliery purposes was for the foundation of a large Cornish pump. This was not satisfactory, owing to want of knowledge on the part of the builders. Later, concrete has been extensively used for engine foundations, in some cases increased strength being gained by imbedding old wire ropes, etc. As a material for dams it is excellent under a slight head of water. As ordinarily built it is too porous for heads exceeding 100 ft. I have observed, however, that if the material be evenly and firmly rammed, so as to leave the minimum of pore space, its resistance to the

* Inspector of H. M. Mines.

passage of water is greatly increased. Under such circumstances the stone should be small and thoroughly worked in.

As a puddling around shafts above the rock head, it is superior to clay, as it is not worn away by the action of any running water. There are other colliery uses to which it is applicable, but I will not here refer to them, as it is sufficient to say that in Nova Scotia it has earned its footing, and that Dr. Murphy has received much commendation for the ready and adroit use he has made of it to extend the funds available for our bridges. This is of no small importance, our rivers being numerous and necessarily bridged near the outlet, as the settlements are principally along the shore.

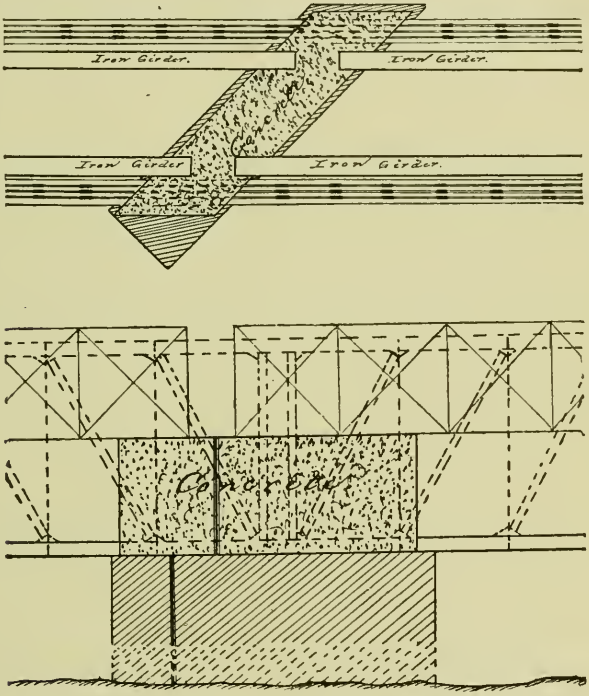
Mr. P. S. Archibald.—The writer is familiar with the various structures described in the author's paper, and can bear testimony to their efficiency and durability. He acknowledges that he had serious misgivings as to the advisability of using concrete for substructures in a climate with such a wide range of temperature as that of Nova Scotia, when the author began some of the works above described. He has, however, had his doubts removed, and for the past five years has been using concrete freely for miscellaneous works in connection with the maintenance of the substructures of the Intercolonial Railway.

In renewing bridges, where it has been found necessary to carry up the height of the piers and abutments, it has been used to great advantage as regards saving of time in the execution of the work, and at about one-half the cost as compared with ashlar masonry.

As an illustration, the writer will briefly describe the renewal of the Rivière-du-Loup Bridge. The old bridge consisted of three wooden spans of 100 ft. each, built on stone piers in a very rapid stream. The trusses were of the "Howe" pattern (deck) and 20 ft. in height. It was decided to replace these with steel spans of the Warren type, 10 ft. deep. This necessitated the building up of the piers and abutments 11 ft. It was the intention at first to build up with solid masonry. The old structure was unusually wide, the piers and abutments extended a very little distance outside of the trusses. The stream was very full, and it was only possible to reach the piers and abutments by lowering the stone with a derrick from the top of the bridge. The traffic over the bridge was very considerable, and it was not practicable to have a derrick set up any length of time on its top. So, under these circumstances, it was decided to try concrete. The old trusses were spread

out a few inches, sufficient to permit of the new spans being erected between them. This brought the old trusses nearly to the outer ends of the abutments and piers.

The chords of the old trusses were 30 ins. wide and took up a large share of the piers and abutments, and in building up the concrete the temporary framework was made large enough to cover nearly the whole surface of the piers and abutments, including the space occupied by the old wooden trusses. These latter were therefore encased in concrete,



as shown on sketch herewith. When the whole was completed and sufficiently hardened, and the new spans erected in place, the old wooden chords and posts were sawn off close up to the face of the concrete, and allowed to fall into the stream below. The old timbers when encased in the concrete were sound. They were chiseled off about ½ in. deep from the face of the concrete, and a coat of cement applied. This concrete has been in use three years and shows no signs of deterioration to date. The cost of it was about $8 per cubic yard. The writer is satisfied that solid masonry would have cost nearly, if not

quite, double that amount. The circumstances, however, were very favorable for the use of concrete; abundance of good clean sand and gravel was to be had near by, while no suitable stone could be got nearer than 200 miles without opening a quarry especially for this purpose. The cement used was the best Portland, and cost, delivered, $2 50 per barrel.

In the past five years the writer has used concrete quite largely in the foundations of turn-tables and miscellaneous machinery, also in bridge substructures and culverts, and in not a single instance has he had a case of failure; even for the foundation of track scales he has found it expedient to use it in preference to masonry. It requires little skilled labor, and no expensive hoisting plant such as is necessary for masonry. The price of good Portland cement is now about $2.10 per barrel, delivered at St. John or Halifax, and when the other ingredients are to be had at reasonable prices, near the site of the work, concrete can be built for less than $5 per cubic yard.

Three years ago the writer had the piers of the Shubenacadie River Bridge treated to a coating of concrete within a timber framework, and has had no trouble with them since. Previous to that, they had been a source of annoyance and expense for 20 years. The masonry here was affected by the flow of the tides, as was that of the Avon Bridge, described in the paper.

Mr. H. S. Poole.—I have knowledge of some of the bridges to which Dr. Murphy refers, and can endorse what he says of the apparent success of concrete foundations, even when made by workmen without special experience. A casual examination readily shows imperfections in the mixing and ramming of the material used and yet no indications are seen of injury from frost, although the climate of Nova Scotia is a trying one, alternations of temperature being numerous, rapid and severe during the winter. I have known a drop of 40° in 12 hours, following a rain storm which left all open structures soaked with water.

Martin Murphy, M. Can. Soc. C. E.—The author attributes the successful employment of Portland cement concrete in the erection of the substructure of the bridges in Nova Scotia largely to the means adapted to carry the work into effect. At first or early stages of progress, the work to be executed was of such a comparatively limited and local character as to be within his own personal control, and was ac-

cordingly let by contract. Later on, however, as the value of concrete became better known and gave a greater degree of confidence, it assumed larger proportions and became more diffused or extended in its application as the bridges under construction became more remote. Hence it was considered that more accurate supervision and more desirable results could be obtained by performing the erection by "day's labor" under the more direct control of the engineer and such assistance as he could procure.

The daily increasing knowledge which regulates the practice of Portland cement, its admixture with the aggregates to form a reliable concrete mass and its final deposition can, in the opinion of the author, for obvious reasons, be better effected by "day's work" or men employed by the day than it can be by contract. Unlike masonry, we have to deal with a material every fragment of which has to be compounded by the union or mixture of elements, on the combination and coherence of which will depend its value and efficiency. The materials have to be selected and their treatment watched with a much greater degree of care than that required for masonry supervision, and this can be better performed by trained hands in the employment of the engineer. The author believes that the failure of Portland cement concrete is more frequently caused by the defective manipulation of the ingredients than by inferior cement. In his experience he has, however, invariably made use of the cement manufactured by White Brothers, of London, England, and its behavior under varying conditions of employment has so far warranted the further continuance of its adoption, and although the other ingredients have not always been as clean, sharp and promising as he could wish to form a reliable concrete, still the structures spread over the Province of Nova Scotia are remarkably free from deterioration or fracture. One thing he is quite certain of, *i. e.*, that, wherever an abrasion or slight flaw shows itself, it is due to careless workmanship and not to the quality of the cement. However, not to be invidious, he should state that he has observed some other brands of Portland cement employed within the Province and equally exposed to tidal fluctuations and to the climatic changes of temperature, already noted, behaving equally well.

The discussion calls for no extended remarks. The gentlemen who have favored us with their views speak from personal observation and from a knowledge of such work gained by experience. In response to

Professor Butler's interrogation, the author has had opportunity, after lapse of time, to examine the concrete deposited under sea water and found no deleterious effects—nothing remarkable, no expansion, disruption or evidence of blowing could be observed. With regard to the voids in the circular piers, the tides rise and fall during ebb and flow, within the same as without. Provision is made against the danger suggested by placing air holes in the top covering, to allow the air forced by the rising of the water to escape.

The properties of a good reliable cement are: It should be finely ground. It should have a specific gravity of not less than 3 at the end of four weeks. It should give the usual tensile strengths with briquettes at 7, 14 and 28 days made with neat cement, and also with briquettes made of 2 or 3 parts of fine quartz and sand and 1 of cement. It should be mixed with a sufficient quantity of water, about 4 galls. to 1 cu. ft. of cement. When too much water is used in the intermixture, a white creamy-looking substance rises to the top of the concrete layer, when tamped. This thin film of free lime more or less divides the mass, separating the continuity of bonds and stratifying the whole, thus destroying its monolithic character. This too frequently occurs by reason of dousing the ingredients with too much water, to save labor in mixing. The author does not believe in excessive tests, more especially tensile tests. A poor cement with an excess of lime may give the best tensile test at seven days, but the induration will not continue steadily onwards with such a cement. One can with some experience determine an excess of lime by the pat test. It is difficult to test cement properly, and a novice cannot possibly do it. The most reliable course to follow is to purchase from the most reliable manufacturers. Extremes of temperature, hot or cold, should be avoided. In summer time with 80° Fahr. and upwards, the concrete should be covered from exposure to the sun and kept sprinkled for six or seven days with a fine sprinkler. In the fall or winter season concrete work should be discontinued in a climate like that of Nova Scotia. where there is risk of freezing. In every case where concrete was placed in piers during frosty weather, when the temperature of the atmosphere was 10 or 15° below the freezing point, the following summer thin scales of concrete would peel off from the exposed surface.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

671.

(Vol. XXX.—December, 1893.)

SUBAQUEOUS FOUNDATIONS.—DISCUSSION ON PAPER No. 626.*

By C. O. GLEIM and F. NEUKIRCH.

A MEMBER.—Has this process been tried, and if so, where?

C. O. GLEIM, Cor. M. Am. Soc. C. E.—The gentleman has suggested just the same thing that I would have suggested, that the author be asked for details of the cases in which he has applied his process. The paper mentions two cases, the one a sewer that had given trouble, being in quicksand, the quicksand being drawn through the masonry and causing a settlement of the surrounding mass; in this case, he says, they have applied the process to the sewer with good success. In the other case he does not give much detail in this respect, and I would like to have the author given an opportunity to add some information on this head.

Mr. F. NEUKIRCH.—In compliance with the desire expressed I beg to give the following details about the practical cases of the execution of foundations according to the method described in my paper.

* "Improved Method of Constructing Foundations under Water by Forcing Cement into Loose Sand or Gravel by Means of Air Pressure." By Fr. Neukirch, C. E., of Bremen, Germany. *Transactions*, Vol. XXIX, page 639.

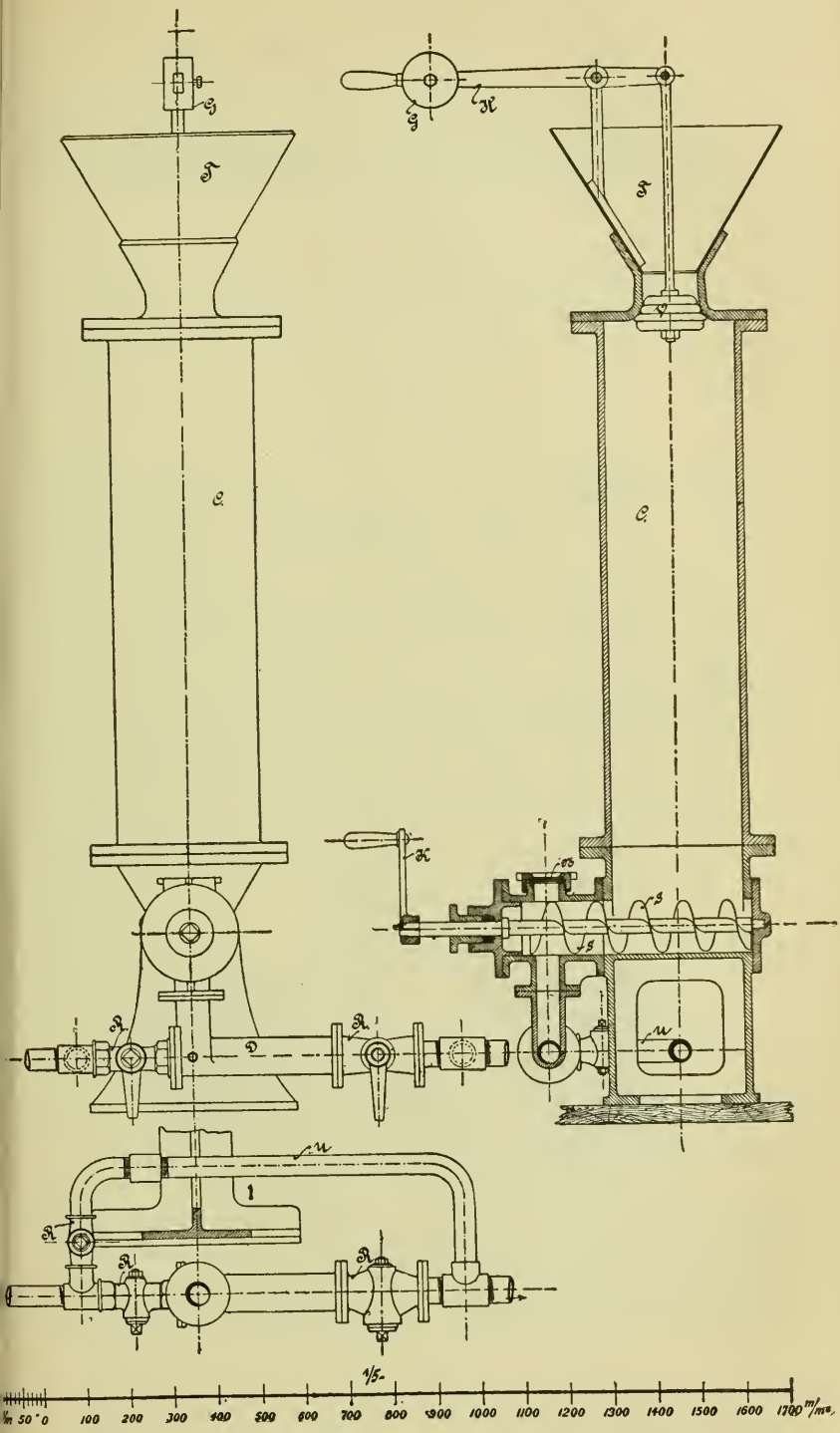
For the work described in Figs. 1 and 2 (Plate I, Vol. XXIX, page 642), a double-acting compressor with direct steam action and increased effect through pressure-compensating slider, patent Burckhardt & Weiss, was employed.

The air cylinder was 270 mm. ($10\frac{5}{8}$ ins.) in diameter.
The steam cylinder " 320 " ($12\frac{5}{8}$ ") " "
The stroke of both, " 305 " (12 ") " "

The compressor forwarded 4 cu. m. (140 cu. ft.) of air per minute, working with 125 revolutions. The maximum air pressure was 6 atmospheres. The air cylinder had a water-jacket, and the compressor was provided with a small pump for the cooling water. The air vessel was 1 m. in diameter, 2 m. long, and was provided with a manhole, a safety valve and a pressure gauge. For producing the steam, the boiler of a portable steam engine that happened to be at hand was used. The compressed air was conveyed to the place of operation through wrought-iron pipes of an internal diameter of 50 mm. (2 ins.). The system of pipes was lengthened or shortened, according to the situation of the place of operation, and was connected by rubber tubing with the traveling stage. In the process shown in Fig. 1, there were 46 580 kg. (102 690 lbs.) of cement injected, and in that shown in Fig. 2, 47 200 kg. (104 000 lbs.).

The apparatus used for supplying the cement is shown on page 581.

The cement is filled in through the funnel *T*, passes the valve *V* and falls into the cylindrical chamber *C*. The valve *V* can be opened by the workman who fills up the cement by means of the lever *H;* it closes spontaneously by the weight *G*. Below the chamber *C* there is a conveying screw *S*, which can be moved by the crank *K*, and serves to convey the cement to the injection device *D*, in which the cement is carried away by the current of compressed air and conveyed to the lance pipe. The pipe *U* serves to convey compressed air direct to the lance pipe without conducting it through the injecting device containing the cement. For this purpose the three cocks *R* are provided. The receiving chamber *C*, which can be shut up by the valve *V*, is required because the holes in the nozzle of the lance pipe might possibly become clogged, and in this case the compressed air would enter through the openings of the injecting device into the chamber containing the cement, which, if the chamber were not closed by the valve *V*, would


1/5.
m 50 0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500 1600 1700 m/m

be blown into the air. At *N* there is a little glass window, to observe the moving forward of the cement by the conveying screw *S*. The chamber *C* is filled every time with such a quantity of cement as is required for each introduction of the lance pipe into the soil.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

672.

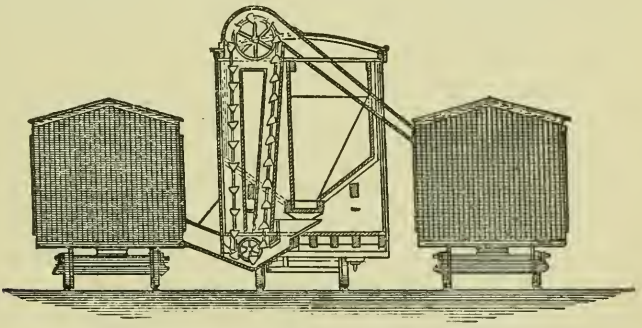
(Vol. XXX.—December, 1893.)

AMERICAN GRAIN ELEVATORS.—DISCUSSION ON PAPER No. 627.*

By EDWARD B. TAYLOR and E. LEE HEIDENREICH.

EDWARD B. TAYLOR, M. Am. Soc. C. E.—I note that Mr. Heidenreich has made no mention of the "Jumbo" grain weighing and transfer car which has been used at Chicago on several of the roads for the past eight years with very good results.

The first car was used on the Pan Handle tracks at Chicago in the summer of 1883, and consisted of a car 35 ft. in length and 15 ft. high



above top of rail. Inside the car, at one end, there was an upright 10-H. P. engine, boiler, and the necessary machinery for a running line of elevator buckets and shovel attachments, the balance of the space in

* "American Grain Elevators." By E. Lee Heidenreich, M. Scandinavian Eng. Soc., of Chicago. *Transactions*, Vol. XXIX, page 644.

the car being occupied by a bin or hopper, of a car-load capacity, resting on a standard scale. In transferring from car to car three tracks are used, as shown in the cut, the transfer car occupying the center track. Steam shovels are used to shovel the grain into an elevator boot at one side and bottom of transfer car, from which it is elevated and dumped into the bin or hopper, then weighed, after which a slide in the bottom of the hopper is opened, and the grain runs a second time to the elevator boot, and is elevated to a spout, by which it is discharged to the car in readiness to receive it. To operate one of these cars requires the services of an engineer and four laborers.

The first elevator built had only a capacity of about 40 000 lbs., but the elevators now have a capacity of 1 800 bushels and a scale capacity of 80 000 lbs. with a 15-H. P. engine, and from 35 to 40 cars of grain can be transferred in 10 hours. The machine has been found to be very useful and economical and well adapted to localities where the business does not warrant the construction of an elevator building.

Mr. E. Lee Heidenreich.—Referring to Mr. Taylor's remarks on my paper on "American Grain Elevators," I will say that in addition to the grain weighing and transfer car which he mentions, and which I often have seen, I also omitted to mention the transportable corn shelling and transfer car which is used to take care of corn from a string of corn cribs along the railroad line. From foreign correspondence I have been reminded of the fact that automatic weighing machinery is largely in use in foreign elevators, and that in an elevator in Fiume, Croatia, electricity is used in transmitting power to different elevator legs, but without any material success. Automatic scales have been used in this country, but have not been brought to a satisfactory degree of perfection, as a possibility always remained for clogging up the inlet or discharge opening of the receptacle by sticks or similar objects which would interfere with the proper workings of the scale. The introduction of a stiff hair brush at the inlet opening seemed to have obviated this trouble in the modern foreign automatic scales. Foreigners claim to have solved the fire-proof bin construction problem by a compound construction in concrete and steel, claiming that the concrete does not cause the condensation of moisture in the same degree as iron or steel; in fact, does not dampen the grain at all, but, in view of actual experiments in this country with cemented brick walls, we cannot consider the problem solved in a manner satisfactory to our grain merchants who are subject to the very strict rules prevailing for the weighing of grain.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

673.

(Vol. XXX.—December, 1893.)

PAVING BRICK.—DISCUSSION ON PAPER No. 628.*

By DANIEL W. MEAD, WILLARD BEAHAN, F. A. CALKINS and ALBERT SMITH.

A MEMBER.—I would like to ask Mr. Mead if all paving brick are vitrified.

D. W. MEAD, Assoc. M. Am. Soc. C. E.—I believe that many good paving brick that are in use in the United States can hardly be classed as vitrified brick. Among the first paving brick which were used in this country that I have seen are those at Bloomington, Ill., which were made in 1872. These were made from the drift clay, and in the same way that the ordinary building brick are made at that place, that is, in "clamp" or up-draft kilns. They are, however, the "arch brick," and I think that from 30 to 40% of the brick nearest to the fire arches are used at that place for paving purposes. These are hardly to be classed in the ordinary acceptance of the term as vitrified brick. Yet they have been in use 20 or 21 years at Bloomington, and are still in fairly good shape, although some have lost an inch or more in the original height of the brick.

On the Ohio River, in many places, they are using an unvitrified

* "The Manufacture and Use of Paving Brick." By Daniel W. Mead, Assoc. M. Am. Soc. C. E. *Transactions*, Vol. XXIX, page 653.

brick. They are made from a clay which is a poor grade fire clay, and the brick resemble largely in color a fire brick, though in texture they are somewhat finer grained and less porous. These brick are ordinarily classified as a vitrified fire brick, which is, however, an erroneous classification, as a fire brick certainly cannot vitrify and still be classed as a fire brick. These are used quite extensively in paving through Pennsylvania, Ohio and Indiana.

WILLARD BEAHAN, M. Am. Soc. C. E.—Is vitrification necessary, or can a good paving brick be made without vitrification?

Mr. MEAD.—We are not sure that brick must be vitrified in order to make the best pavement, but it must be a first-class hard-burned paving brick if we expect to use it successfully. There is considerable doubt in my mind whether this idea of using none but vitrified brick is a safe one. If we vitrify brick it is not so susceptible to the action of frost, but we are apt to make a brittle brick which will deteriorate in a few years. In regard to vitrification, I would say I do not regard vitrification essential to all paving brick, at least in a great many classes of pavements that receive the lighter traffic. In the heavier traffic, however, and in the higher grade paving brick, it is still an open question as to what the best qualifications for such a brick are.

F. A. CALKINS, M. Am. Soc. C. E.—The tables contained in this paper comprised in Nos. 2–4 and 8, are of value as reference for analysis, method of manufacture and details of laying pavements. The suggested outline for specifications will tend to increase the efficiency of the limited specifications now in general use.

The principal evidence now required for brick pavements are practical tests for endurance under heavy traffic, which time only can supply. To that end the following description of a brick pavement is contributed. This pavement, although young in years, is old in experience, when compared with ordinary brick pavements, is located in Chicago, on South Canal Street, near Harrison, in the freight yard of the Chicago, Burlington and Quincy Railroad Company, and was laid in June, 1890.

This yard is in the form of a parallelogram, 570 ft. in length and 40 ft. in width, with the longest side parallel with the street. The freight house is 555 ft. long, provided with 17 doors for reception and delivery of freight, with a space of about 31 ft. between each. The entrance to this yard is on one side, and not at the end, and is about

400 ft. long, or 155 ft. less than the length of the freight house. It is important that the position of the yard relative to the street be noted, to understand the effect of traffic.

The foundation for this pavement has been a cumulative one, composed of broken stone which has been added to for many years, as the settlement under traffic required; and at time of laying the brick, it had a depth varying from 18 to 24 ins. Upon this foundation a cushion layer of sand was spread from 4 to 6 ins. in depth, upon which one course of brick was laid on edge, at right angles with both the street and freight house. This position of the courses turns a large portion of the traffic parallel with, instead of at right angles to, the paving blocks.

As a team is in position at each door nearly all the time, the space to turn is limited, compelling the driver to turn short and back the wagon for delivery of loads at the end and not from the side, which produces a twisting motion to rear wheels causing them to slide and not roll on the pavement. As this rolling motion is mainly parallel with courses, it tends to increase the destructive action to a greater extent than a uniform rolling load would be at right angles. The traffic on this pavement during the past three years has been heavy.

Men were stationed to count the teams entering and departing from the yard in one day. The tonnage of freight was given by the company, but the weight of teams, wagons and drivers was estimated. The total amounted to 11 157 tons, or 279 tons per square foot of surface. This computation conforms with Captain F. V. Greene's tables.

A recent inspection of this pavement shows that it is in good condition, presenting a fair surface for a pavement under such a heavy traffic, yet there are defects which a careful inspection shows were caused by poor material. All of the bricks are chipped on the upper surface, leaving a smooth surface from 1 to 2 ins. in width. The vertical chipping was from $\frac{1}{8}$ to $\frac{1}{2}$ in. on sound brick, and on some soft brick, 2 ins., yet this chipping does not prevail to such an extent as to impair the usefulness of the pavement. I noticed a few depressions which are produced by soft bricks. There are five depressions about 1 ft. in diameter which were from 1 to $1\frac{1}{2}$ ins. below surface, and one 6 ft. long and 1 ft. wide, 1 in. below the general surface. The bricks at each point could be cut easily with a knife. The adjacent bricks presented a uniform surface.

The forces causing destruction to this pavement appear to be due more to percussion than compression. The greatest amount of chipping occurred at a point where the heaviest traffic passed, at entrance, which would be longitudinal with layers. The next in importance was noticed at points where the horses turned in backing, which coincided nearly with position when at rest during loading.

If the soft bricks were replaced with sound ones, this pavement will endure present traffic many years longer.

The bricks composing this pavement were made in Galesburg, Ill., but are generally inferior in quality to the paving bricks I have examined from that locality.

ALBERT SMITH, Assoc. M. Am. Soc. C. E.—Mr. Mead's paper will be welcomed by municipal engineers as a valuable addition to the literature of an important subject. No doubt many will differ with him in the opinion that "brick will eventually outrival granite under the heaviest traffic," or at least render the Scotch verdict, "Not proven"; but its value where the traffic is moderate, as in small towns and cities and the residence sections of large cities, has already been demonstrated. I would not be understood as advising its indiscriminate use in all such places.

It is at all times the duty of the engineer to secure the best results for the least money, and while in some places the price of good paving brick is such as to make this the natural material, there are other places where one may well question the wisdom of its use, the added freight from many hundred miles of railroad haul increasing the cost above that of other equally good material which can perhaps be found near at hand.

The qualities which go to make a good brick pavement are essentially the same as for a good pavement of any other material. The brick form the surfacing material and must resist the abrasive action of the wheel and the horseshoe, and the disintegrating action of water and frost. The comparative ability of different brick to resist such abrasion is fairly shown by the rattler or tumbler test. Although a rattler can be found at any foundry, it is difficult to find two of the same size or run at the same speed; therefore, make a comparative test with some well-known brick or paving stone. You will certainly do well if you can attain the standard of quality set by F. A. Calkins, M. Am. Soc. C. E. (*Transactions* Am. Soc. C. E., Vol. XXVI, page 391),

viz., first quality, abrasion equal to granite; second quality, abrasion twice that of granite.

Brick should be tough enough to stand dumping from wagons; if they do not stand this they will fare badly under the chipping action of a horse's shoe. Brick must also be able to resist the action of frost, their ability to do so varying in a general way with their porosity; but being also a function of their elasticity or toughness, it seems probable that an amount of expansion from frost which would result in the disintegration of one brick might not produce strains beyond the elastic limit in another brick composed of different material. My experience has been that a maximum limit of 4% absorption in fire clay brick and 2% in mixed clay or shale brick, can be met by the manufacturer, and that within such limits little trouble is to be apprehended on that score. A prime requisite is uniformity in mixing, molding, and burning. All clays must be thoroughly rolled or ground, and should be molded so as to be as free as possible from lamination. Great care is necessary in the burning. If the brick are raised to too high a heat before the "water smoke" is driven off, various defects arise. First, cracked or "checked" brick; second, "scalded" brick, showing a discolored core, generally blue; third, "bloaters," or swelled brick. The next stage is the burning proper, and must be continued long enough to burn the brick clear through. A failure to do this will leave an unburned central core differing in shade, but not in color, from the outside surface. A brick with a core, either from scalding or under burning, is necessarily an imperfect brick. Not only is there a failure to develop the full strength of the material, but the lack of uniformity necessarily induces strains within the brick itself, thereby tending towards its own destruction.

The third stage is the annealing or slow cooling. If this is properly done, the brick, although vitrified or glassy in structure, will have a proper degree of toughness. If cooled too quickly, they will be brittle, a great many will be damaged in handling, and they will be liable to chip up under the action of the horse-shoe.

I see no advantage in round-cornered brick, nor is the repressed paver, as ordinarily put on the market, superior to the ordinary paver in aught but shape. The pressure given is only enough to true up the brick, but not to compact it to any appreciable extent. They stand handling somewhat better and lay closer together in the street, but, so

far as I have tested them, are not quite as strong as the unpressed brick made by the same manufacturers. Apparently the superficial layer affected by repressing defeats the very object aimed at, *i. e.*, uniformity of structure. These remarks would not apply if the pressure were greater. The immense pressures put upon electric light carbons, emery wheels, corundum wheels, etc., certainly have a good effect. These pressures run up as high as 2 tons per square inch, and thus assure a uniformly dense material.

Austin T. Byrne in his excellent book on highway construction (page 152) says: "The foundation should in all cases be formed of cement concrete," and some others hold the same opinion. One might as well say that all railroads should be double tracked and laid with 90-lb. rails. There is no doubting the fact that a good concrete foundation will make a better street than any fragmentary construction, but there are many places where the traffic can be sustained on a much cheaper base (as witness Table No. 8, given by Mr. Mead).

In this list nine towns and cities are reported as using concrete wholly; four use it to some extent, while thirty use fragmentary materials according to their local availability. Personally I have known very good results, both from gravel and broken stone foundations; but have had no experience with a sand foundation, either in connection with tarred boards or an additional course of brick laid flatwise.

A question frequently asked is, "On how steep a grade can brick pavement be used with safety?" The steepest instance within my personal knowledge is that of Main Street, Washington, Penn., where a maximum of 11% is reached and no trouble experienced.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

674.

(Vol. XXX.—December, 1893.)

PRINCIPLES FOR THE LAYING OUT OF CITIES. DISCUSSION ON PAPER No. 632.*

By JAMES OWEN.

JAMES OWEN, M. Am. Soc. C. E.—There are two points about Mr. Stübben's paper which are very forcible and striking: 1st, that the theoretic principles enunciated therein are so in accord with the ideal practices aimed at, but perhaps not adopted, in this country; and, 2d, that the writer, with the surroundings of European practices, where haphazard extension from an accidental nucleus is so much in vogue, should have been enabled to divest himself of the influence of environment and so enunciate the doctrines that he has.

In a new country, such as this is, the possibilities of ideal mural development are infinite and unfortunately seldom realized; in Europe such development is in spite of tradition and the habits and ideas of centuries.

There are, however, principles enunciated by Mr. Stübben in his paper that, with the present ideas of city transit and with its possible future development, seem open to criticism, and I wish to call particu-

* "Practical and Æsthetic Principles for the Laying Out of Cities." By J. Stübben, Baurath and Asst. Burgomaster at Cologne, Germany. *Transactions*, Vol. XXIX, page 718.

lar attention to the matter, as, with the experience in large cities for the last few years for the proper service of vehicular and traction travel, a partial abandonment of old ideas seems necessary.

In the laying out of any city two main factors enter into consideration —suitable location for dwellings and buildings, and ease of access from and to these buildings for the general population. It therefore follows that, considering only the dwellings themselves, a rectilinear system is all that is required, making all blocks square, and, of necessity, all buildings the same. As against this is the desire of the traveling public to get to its destination in the most direct line, and the rectilinear system does not, on the average, allow this. As a relief to the necessity of traveling two sides of a triangle, radiating streets are introduced, and, in many of the cities of this country, are in use, and in a few cases, like the city of Washington, a series of intersecting radial avenues from different nuclei were constructed. This, while thoroughly accommodating the traveling public, sacrifices the land for building purposes, leaving many goring lots and impairing their value.

With the growth of rapid transit and the cheapness of fares the necessity for radial avenues decreases in a marked degree, in consequence of the ability of more people to ride than previously; and so, only such radiating streets as are necessary for the rapid transit should be constructed, and, by that, more of the rectilinear system preserved. In laying and constructing such main arteries, new ideas will have to be in vogue to properly subserve a general interest. As it is now, we have the riding, driving and walking public to consider. All have their rights, and all should be considered. So far the walking public has been unmolested. The driving public now, with cable and trolley roads, is driven out of the main avenues of travel, and the riding public is supreme.

An inspection of the condition of the streets of Chicago at the present time will illustrate the force of the statement; trains of cable cars careening through the main streets of the city, ruthlessly forcing aside all other travel on it, do not constitute a condition that may be considered ideal. Take the city of Boston, in its present congested condition near the Common, and vehicular travel is almost out of the question.

It may be urged that such a condition is tentative, and that a proper solution of the rapid transit problem in all such cities will

remove the congestion. In answer to this, the reply is that, so far, human invention has not been able to keep up with demands of locomotion in cities, and, as long as their population keeps increasing, will probably not do so.

The ideal street in the future must therefore consider all requirements—so much for foot travel, so much for vehicles standing, and so much for vehicles moving, and so much for cars that are propelled. To accommodate all, more room than has as yet been considered requisite must be allowed. The extension of Beacon Street, Boston, into Brookline is an instance of about what is necessary. It is 150 ft. wide, and about fills present requirements, and in any new city development a width of main arterial street less than 150 ft. should not be considered.

There are many other points in Mr. Stübben's paper worthy of allusion, but I merely wish to call particular attention to what I consider a vital point.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

675.

(Vol. XXX.—December, 1893.)

MANUFACTURE AND TESTING OF PORTLAND CEMENT.—DISCUSSION ON PAPERS Nos. 633* AND 634.†

By D. J. WHITTEMORE, R. W. LESLEY, WILLIAM MURRAY BLACK, Sir BENJAMIN BAKER, WALTER ROBERT KINIPPLE, JOHN PURSER GRIFFITH and HENRY FAIJA.

D. J. WHITTEMORE, Past President Am. Soc. C. E.—While not prepared to discuss the able papers under consideration, I wish to call your attention to the elaborate paper written by Captain Maclay and read before the Society at a recent meeting, advocating certain severe tests not generally advised either in England or Germany. My own opinion is, that the boiling is of little or no value in determining the ultimate quality. The effect of immersion in warm water is to increase activity of induration; but when immersion takes place in boiling water, the steam generated in uncombined water in the immersed specimen may, and probably does, produce a disruptive effect upon the best cements. American engineers wish to secure a uniform cement, and one that is finely ground. It is, I think, a question whether we should test ce-

* "The Testing of Portland Cement and the Development of the Cement Industry in Germany." By Max Gary, C. E., Berlin, Germany. *Transactions*, Vol. XXX, page 1.

† "On the Manufacture and Testing of Portland Cement." By Henry Faija, M. Inst. C. E. *Transactions*, Vol. XXX, page 43.

ments by compression, and not rely entirely upon tensile tests. If we wished to ascertain what a man could sustain on his shoulders, we would not attempt to ascertain the tensile strength of his neck. Cements are generally used in compression, and more tests of that nature should be had. While the ratio between the tensile and compressive strength of Portland cement appears to be practically constant, this is not the case with many of our natural cements of America.

In conclusion, I wish to say, that it is my belief that the quality of Portland cement has not materially changed or been improved since the close of the investigations of the late John Grant, M. I. C. E., were published. As I now remember it, he established that the cement he used in the metropolitan works of London should withstand a tensile strain of 250 lbs. per square inch at the age of seven days, the last six being in water, while it is a common practice in this country to exact a tensile strength of 375 lbs. per square inch. In explanation of this apparent difference in the demands, with, as I believe, no great difference in quality, I wish to say that Grant's tests were made on specimens having a breaking area of $1\frac{1}{2} \times 1\frac{1}{2}$ ins., or an area of $2\frac{1}{4}$ sq. ins.; while tests, both in England and America, are now made on briquettes having an area of 1 sq. in. Perhaps it will be borne in mind by some of our members that in a paper read before this Society by myself some years ago, it was shown that briquettes of the same cement, having breaking sections varying from $2\frac{1}{4}$ to $\frac{1}{2}$ sq. ins., the unit of measure of strength was not the area, but the periphery, of the breaking section, that is to say, that the total strength of briquettes of different sizes, but of the same cement, when divided by the length of the circumscribing line of the breaking section, gave approximately the same resulting quotient. This being the case, then Grant's requirement of 250 lbs. per square inch for $2\frac{1}{4}$-in. area of breaking section would be as severe as a demand of 375 lbs. for a section of 1 in. square. This being the case, is of itself a strong argument for the establishment of a universal standard briquette.

To some considerable extent the same rule holds good in the tests of stone, brick, glass, and other substances of a brittle nature, when tested in short lengths.

R. W. Lesley, Assoc. Am. Soc. C. E.—In discussing the two papers which have just been read, and the comments that have been made upon them by Mr. Whittemore, there seems to me to be quite a number of

matters that are of considerable interest, and I should like to call attention to several of them.

In the first place, upon the question of hot tests, I would say, commenting upon what has just been said by Mr. Whittemore, that he is entirely right in his assertion that the paper of Captain Maclay has set forth a violent test for Portland cement, and one which controverts the practice of the Germans and also the leading authorities in England. This is not only the case so far as concerns the first consideration of the subject by the Society of German Cement Manufacturers, but is also the case as to their later tests. The question has been the subject of their consideration for three years, and in the last Proceedings of that Society, of which Mr. Gary is a member, the Association has just resolved, after further investigation, to adhere to its regular cold water test and to dispense with any boiling test. These results have been arrived at after a series of tests covering a period of nearly five years, and are the final conclusion of the leading society of this character in the world.

In connection with this German society, which in some of the papers upon the boiling test has been stated to be governed in its adoption of tests by the "financial interest," as though it were an association of manufacturers only, it may be stated, and its proceedings show, that at all the meetings the German government is represented by its Director of Testing Department for Building Materials, as well as by representatives from public works, post office and other departments of the German government, and that furthermore the standard tests adopted by that society are recognized by the Minister of Public Works in Germany as the official test in that country.

The opinion of Mr. Faija, who is one of the leading authorities in England upon this subject, also practically sustains the same view. His test is a very simple one so far as any heat is concerned, and it suggests that the pats of cement, when made, shall be put upon a little shelf in a closed tin vessel, where moisture is arising from water kept at a temperature of about 116° Fahr., and that, after a few hours of this exposure, these pats shall be placed in water at about 116° Fahr., and shall, after 12 to 15 hours of this treatment, be examined for checking. The effect of this test is almost the same as covering briquettes with a damp cloth in sunlight, and is certainly a test that is efficacious so far as determining the quality of cement is concerned, while on the other

hand it is not destructive, as are the accelerated tests in the various papers read on this subject by Captain Maclay and other cement chemists, as distinguished from engineers. In order to determine the effect of the test described by Mr. Faija, and of the method described by Mr. Maclay, I took a well-prepared Portland cement clinker, grinding it to a fineness to pass 90% on a 10 000-mesh sieve, made the material into pats, and put the pats through both modes—those of Faija and those described by Captain Maclay. The tests show, as I have always claimed, that there is a time in the chemical setting of cement, when, if there is any extraneous influence brought to bear upon the cement at the particular moment mentioned, the setting is retarded, sometimes entirely prevented, and sometimes driven into violent action.

In the test described by Captain Maclay, four pats are made which are put into the water at various periods. One of these pats is placed in a steam bath, temperature 195 to 200° Fahr., as soon as it is made. The second pat is placed in the steam bath as soon as it is set hard and can bear the 1-lb. wire. The third pat is placed in the steam bath after double the interval has elapsed that it took the pats to set hard, counting from the time of gauging. The fourth pat is placed in the steam bath at the end of 24 hours. The four pats are each kept in the steam bath three hours, then immersed in water of a temperature of about 200° Fahr. for 24 hours each, when they are taken out and examined. Out of these pats which were exposed to tests under this method, none showed any disintegration except the third pat, which had been left in air two hours and then in water; whereas, the pats made with the same cement, tested under Mr. Faija's method and subjected to the slow moist heat, then to water at 116° Fahr, showed no disintegration whatever. From these facts I am of opinion that the accelerated test above mentioned certainly is not as fair to the cement as the test suggested by Mr. Faija. In other words, as we all know, if there is one thing that no man can give an opinion on, it is the exact moment when the cement sets—one can say it can be told by the needle; this needle can be pressed upon the pat within ten minutes after set, or within an hour after setting. Another will say that the set can be told by tapping the cement with the nail—this nail may be long or it may be short, and there is an equal uncertainty about the moment of setting. The whole action of setting is a chemical one, and there is a moment in the chemical setting of cement, as the experiments above referred

to show, when, if the cement is disturbed, it will not give the result; and for this reason, it seems to me that boiling pats at high temperatures will never give absolutely correct results and this test should certainly not be adopted from the authorities so far before us.

As against this, the German report follows out a series of experiments made by the Society, and gives results of these experiments and the effect which followed the boiling test, the record of the cements which had stood all the requirements in water, air and in the work, and yet which had failed to stand the accelerated test. It follows out these results carried out at three years, and also the result of other cements, which had failed to stand the boiling test, carried out to a longer period.

The same matter of accelerated tests has been up before the French Society of Engineers of the Ponts et Chaussées, in France, where it was sought to introduce such a test as one of the elements of the standard specifications for work in that country, and, at a late meeting, the French authorities reported against an adoption of such test, so that upon the broad principle of the adoption of accelerated tests in this country, we are left with the fact that the consensus of opinion of the German government, the French government, and the leading cement authorities in England, together with the suggested specifications of the American Society of Civil Engineers in our country, is all against the adoption of any such test.

It is a very gratifying thing to find that Mr. Gary, Mr. Faija and Mr. Whittemore all agree in their views as to the use of this boiling test, and, as engineers, sustain the views of engineers upon this subject, as against the experimental laboratory tests that have been suggested.

Referring briefly to some of the points in Mr. Faija's paper, his specification seems to be a very fair one, inasmuch as it requires a cement of moderate tensile strength at seven days. This would enable the manufacturer to make fine ground cement, which in the neat tests will not test as high as the coarser ground cement made with the same clinker, but will test high in sand mortar. In the matter of grinding, the American manufacturers are following the Germans rather than the English, and the bulk of American Portland cement will pass 90 to 95% on a 10 000-mesh sieve; results of this cement would show on a 1-in. section about 300 to 350 lbs. at seven days neat, and from 200 to 300 lbs. at seven days in 2 to 1 sand mortar; the sand mortar showing the

sand carrying effect of the fine grinding, while the low figures in the neat test are due to the same cause, which usually retards the neat test.

Mr. Faija's paper gives a very interesting description of the various kinds of machinery in use in England, and, in connection with this matter of fine grinding, he makes some comparisons between stone mills and iron mills. In this country we have solved the problem of grinding cement by the use of iron mills, which certainly grind a larger quantity and give more satisfaction in the manufactured product than stone mills do. We have also overcome the objection to mill stones suggested by Mr. Faija, viz., that of having one stone always up being dressed, because in the cement mills in this country all the mill-stones are made interchangeable, and, by having one extra pair of stones, all the mills may be run and still a pair of stones be dressing, the loss of time being only the moment of taking one pair of stones out and putting in the other.

In our practice here, we have also overcome the difficulty of making the brick too dense for burning, which Mr. Faija states is the difficulty that arises in making brick in brick machines. By our practice we find it is better to take the raw material, and, after pugging, to spread it on floors and then cut it by hand. This makes more porous brick than those made by machine, and, at the same time, makes brick in the best possible shape for burning.

One of the most interesting things in connection with sand tests of cement, and one which possibly few engineers have thought of, is that the effect of the sand test is not only to get the result of a cement, but also to get the result of the sand actually used or to be used in the work; that is to say, while a series of tests of a number of cements with the standard sand of the American Society will give the varying elements of the several cements used, so a series of tests with different sands made with a single established brand of cement will give the comparative results of the various sands proposed to be used. Interesting experiments of this character were made with certain red sand used in the brickwork of the Niagara Tunnel, and tested on a comparative basis with the standard sand of the American Society. Similar tests of the same character have been made on the New York Aqueduct, the majority of briquettes especially referred to in paper on the Sodom Dam, read by Mr. W. McCulloh, being made with sand actually used in the

work, and many others being taken and made direct from the mortar in the mortar-boxes. At one of the Scranton dams, near the city of Scranton, Penn., there was a great deal of trouble about the sand, and work was suspended at one time while the matter was being investigated. The sand at that time was being brought from some distance to the work, and a suggestion was made to test the sand found in a hill near by. As a result of the experiment, all the sand being tested with the same cement, and a similar set of tests being made with the standard sand of the American Society, in which the same cement was used—results were arrived at that showed that the sand near the work was very much better than the sand that was being brought from a distance, and by comparison between it and the sand of the American Society there was very little difference. In this way a large amount of money was saved the contractors in their work.

Some remarks have been made in Mr. Gary's paper in the matter of abrasive tests of cement, and machinery for the purpose of making these tests has been fully described by him. In this country, the question of abrasive tests for cement came up in connection with the Niagara Tunnel, where a large body of water was to be dropped at a certain point in the tunnel, and the best proportioned mortar was desired for use in that connection. Dr. Coleman Sellers took the matter in hand and made a number of experiments, to arrive at the best results in this direction. The method used by him, and which he will no doubt describe in proper time, was by means of a sand blast, and the results of his experiments were very much in the same lines as those shown by Mr. Gary.

Another thing which may be of interest is the difference in the several machines for testing tensile strain. The German machine and the Fairbanks machine are very much alike, being automatic and having shot running into a bucket at a uniform rate of speed, the breaking of the briquette causing the bucket to drop and thus cut off the supply of shot. The bucket is then weighed, and the strain is registered on the scale. The other machines largely in use in this country are the Riehle and Olsen machines. These machines consist of a long beam, at one end of which is the briquette, and at the other end of which is a fixed weight. On the beam on which the pounds are registered is a movable weight. In manipulating the machine, the operator uses one hand to turn the screw below the briquette so as to

tighten it up and with the other pulls the weight along the beam, keeping his eye on the needle so that the strain may be kept even throughout. These machines on long-time briquettes, give better and more uniform results than the shot machines, but on short-time briquettes, owing to the personal equation of the operator, very frequently give incorrect figures. In the testing laboratory of Booth, Garrett & Blair, in the city of Philadelphia, a very interesting improvement has been made to one of these beam machines, in order to make the work of the machine more uniform. In this device a grooved wheel has been substituted for the hand wheel, by which the operator applies tension to the briquettes; a cord runs in the groove and from it around a pulley on another shaft suitably placed. Motion is given to this shaft by means of of a disc sliding on a feather on the shaft. The disc's rim bears on the face of another disc at right angles. The shaft which carries this second disc runs steadily by power, but, by shifting the rim of the first disc across the face of the second, the speed of the pulley-shaft can be changed at pleasure, from fast to slow, or to the dead center. The weight can thus be applied to the wheel at the rate of 400 lbs. per minute, and the operator simply has to turn the small wheel that draws the weight along the scale beam. Briquettes broken on the machine thus operated show a breaking almost uniformly at the central section, and show average results considerably better than on the same machine operated entirely by hand.

William Murray Black, M. Am. Soc. C. E.—I desire to invite the attention of the Society to the necessity for tests for the relative value of different brands of cement for sea work. I was in charge of some work in St. Augustine, Fla. There, you may know, the sea water is very salt and the heat of summer very intense. In 1886 we had to do some work with Portland cement concrete in air and in water. The cement that we had was an English Portland, and stood all the usual tests. The sand that we used was very fine, but a sand that afterwards gave very good results indeed. It was almost entirely silica. The only coarse material that we had there was shell. In that vicinity there are large banks of broken shell, called coquina gravel, and that material was used because stone could not be obtained without too great expense. The proportions used were 1 of cement, 2 of sand, and 4 of shell. The concrete was used in three places; first, in an

arch in air; second, at the foot of a sea-wall, which was exposed at low water; and third, in some blocks which were made for tests, made in air and left to dry in air for a week, and afterwards placed in sea water. The arch in air rapidly became very hard. After two years I was obliged to drill a hole through it, and it was as good concrete as any one could desire. At the same time it became necessary to excavate at the foot of the sea-wall. We found the concrete there very soft. I then took up the test blocks that had been lying near by in the sea and examined them. The concrete of the test blocks and of the sea-wall had become very soft, looked like scoria in texture and the color had changed. I had the matter investigated as well as I could, but could not determine the cause for the change, and could not find any test that would enable me to guard against future deterioration in similar work. The only thing I found it possible to do to guard against such deterioration was to call for and use some brand of cement which had stood successfully under similar conditions for five years. I doubt not that a large amount of concrete of the foundations of sea-walls is practically worthless. As concrete in foundations is subjected to compression strains, and as the strength is generally well within the factor of safety, this may not be discovered. I found that engineers whom I interrogated on this subject generally could not tell me anything about the later state of the concrete they had used under such conditions.

Another instance of concrete softening might well be noted. The concrete foundations of Forts Taylor and Jefferson were laid about 1856. This concrete was made with Rosendale cement, calcareous sand and shell. About 1876 a portion of the sea-wall at Key West was repaired with concrete of the same materials. Portions of this concrete are covered at high water; when the water falls, small puddles are left at any places where it will lodge, and is acted upon by a very hot sun. Along the top of the wall where these puddles are found, you can take a penknife and sink it in the concrete to the length of the blade; along the side of the wall where the water does not stand, the skin is left on, and the concrete is hard. We had another experience when in Florida which shows the strength of concrete necessary to resist abrasion by wave action, in putting in some concrete just at a point where it would be exposed to surf at every high tide. It was placed at low water. The concrete was made in blocks and protected by

strong boxing. About a day and a half after we got the blocks in we found that a portion of the boxing had been torn off during a heavy gale. We had made quite a number of test pieces of the same concrete at the same time, and these were kept under conditions quite similar to those to which the walls would be exposed. We removed the remaining boxing from the block of concrete after a week and found that the surf had cut in in some portions and that other portions of the block were perfectly solid. In testing the concrete test blocks, we found that wherever the concrete had not attained a tensile strength of 30 lbs. per square inch before exposure to the waves, it was abraded; where it had attained to 30 lbs. per square inch before exposure, it was intact.

Sir Benjamin Baker, Vice-President M. I. C. E.—Mr. President, I think that although the literature on this matter would fill this room, still engineers are not in accord upon the subject. The Manchester Ship Canal and the Liverpool Water Works are instances; the specifications require the tensile strength to be at least 80% higher in the one case than in the other. I know both works, and on the ship canal there are some hundreds of thousands of tons of cement. I paid a visit of inspection just before I came over and carefully examined all the great works of the docks and sluices, retaining walls, inverts and locks, and I failed to find cracks, except in some places where they would generally be found, namely, at the outside angles of walls returned at right angles. In the case of the canal works, the cement was of low tensile strength, about 300 lbs. per square inch. On the other hand, in the instance of the great masonry dam of the Liverpool Water Works, one of the largest in the world, the cement was specified to be about 80% higher than on the ship canal. The engineer was entirely satisfied, as there had been no cracks whatever. Both engineers, indeed, were perfectly satisfied with the results. These engineers were familiar with all the literature on the subject, and with the views of English and of foreign engineers, and yet we have that great divergency in practice. Others say we never had any trouble with cement 30 or 40 years ago, when the matter was left to the experience of the manufacturer, and engineers did not insist upon tests.

Of course, in masonry structures the direct tensile stresses are small, but shearing stresses are a very important question, and in many respects you will see that failure by crushing is really a failure by

shearing. In all the many experiments which I have made with concrete blocks and concrete columns, some of them 2 ft. square and 12 ft. long, I found the strength directly proportional to the quantity and tensile strength of the cement, and that the failure generally occurred by a longitudinal split down the column.

There is one point in Mr. Gary's paper which I really rose to speak to. He says, with regard to the setting of cement, "it is best to employ Vicat's regulation needle," which may have been true a few months ago, but is not true now, because an autographic machine is used now, of a simple character, which any instrument maker could make. That is, instead of a needle, a small wheel is used, loaded with a few pounds weight and traversed by clockwork. A pen attached to a lever traces a curve of sufficiently large scale to constitute a permanent autographic record of the rate of setting of the cement from minute to minute.

Walter Robert Kinipple, M. Inst. C. E.—Mr. Faija's paper deals with the manufacture and testing of Portland cement, but engineers, as a body, are more concerned with the cement after it has been manufactured, and it is important that simple, reliable, and, as far as possible, expeditious methods of ascertaining whether cement be sound and fit for use, should be well known.

Mr. Faija has set forth very clearly the usual tests, but there is one addition which I have found to be useful, and have adopted in a number of the works which I have carried out. That test is to form a thin plate of Portland cement 12 ins. square by $\frac{1}{8}$ in. in thickness on the surface of a piece of plate glass, surrounded by a metal border, or framing, standing up $\frac{1}{8}$ in. above the surface of the glass. Through the cement plate, near to one corner, after it has slightly set, a hole is pierced for the purpose of hanging it up by a piece of wire or string.

When the plate has firmly set, it is hung up in the testing-room, and carefully observed, from time to time, to see whether any cracking of its surfaces or edges takes place, or any tendency to twist, and I have always found with good cement which has been duly cooled, that the testing plates keep their form, and no signs of cracking ever appeared; and, further, that the plate, when hung up by a wire or string, gave a clear, musical sound, which I found to be that of E flat, in almost all cases. With cement which was inclined to blow, the plate did not maintain its form, nor did it give forth the musical sound referred to.

I have in my possession some plates which have remained true planes since first formed, while there are others which have gone to pieces or become twisted, to a considerable extent.

By forming such large, thin plates better results are obtained than by the usual round plates of a few inches in diameter, by about $\frac{1}{2}$ in. to 1 in. thick in the center, which I have frequently found do not show defects until a very long period after the plates were made. I have some in my offices at present, which are only now falling to pieces after having been made some eighteen months.

The grinding of cement is most important, and for the application of it, in the form of thick paste or grout of neat cement, for the purpose of cementing together stone or shingle or large blocks under water in the construction of monolithic or submarine work, fine grinding is highly important. To this system of submarine monolithic work I have devoted 35 years' attention, and have carried out many undertakings based thereon. My experience with cement for this purpose shows that it should be very finely ground, and that the ordinary sieve of 2 500 meshes per square inch is not sufficiently fine.

My specification for cement for this class of work is that it should pass through a sieve of 6 400 meshes to 1 sq. in., and for the St. Helier Harbor Works, Jersey, where many thousands of tons of cement were used, this was the specified firmness, and excellent results were obtained therefrom. With cement passing through a 2 500-mesh sieve, some of which was tried, decidedly inferior work was obtained.

A small parcel of cement ground to pass through a sieve of 10 000 meshes to the square inch, was likewise used on these works and gave the greatest satisfaction, and had it not been for the extra price per ton which the makers required for cement of this fineness, I should have had only this fine floury cement used on the work.

Mr. John Purser Griffith.*—Mr. President, I sincerely trust that this paper of Mr. Faija will be fully discussed. The subject is of world-wide importance to our profession, and I believe that this is the first opportunity which we have had of hearing the views of engineers of all nationalities. For almost a quarter of a century I have been engaged in harbor work, and it has been my privilege to be associated with one of our most distinguished British engineers, Dr. Bindon B. Stoney, in the improvement and maintenance of the harbor of Dublin.

* Of Dublin, Ireland.

We have used Portland cement largely and with great success, and it will readily be understood with what interest and anxiety we have heard of some of the recent failures which have occurred in marine works constructed with Portland cement. I have endeavored to investigate the causes of these failures, and I am glad to be able to say that in the majority of cases they have been due to the manipulation of the cement, and not to any fault in the cement itself.

We have had in England a magnesia scare, as it was alleged that the presence of magnesia in the cement was the cause of many failures, and it has been proposed to limit the proportion of magnesia present in Portland cement to a very small percentage. It is now generally admitted that the presence of large quantities of magnesia in the concrete where these failures occurred was due to the percolation of sea water, which had decomposed the cement, and that the remedy, or rather the preventive, for such a state of affairs is to make the concrete impermeable.

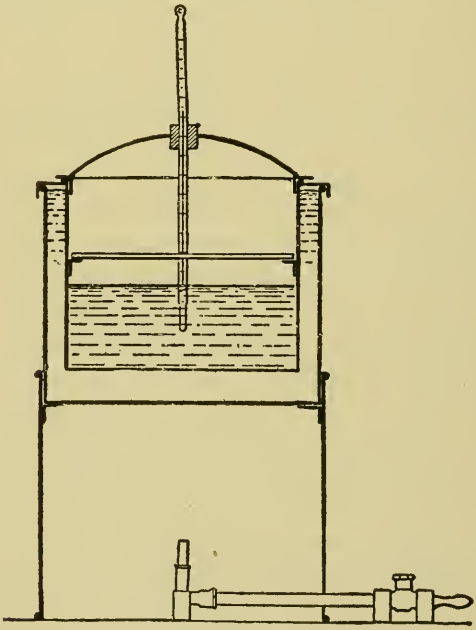
Germany has adopted standard tests for cement with very great advantage, both to consumers and manufacturers. I believe that Germany owes much of its success in the manufacture of Portland cement to these tests, and I am indebted to my friend, Mr. C. O. Gleim, of Hamburg, for much information on this point. In England, unfortunately, we have no standard tests, and the result is that engineers there have to be more continually on their guard against dangerous cement. That such dangerous cements are to be met with in our markets, I have had ample proof. One instance in particular comes to my mind. It is the case of a cargo which passed the specified tests for tensile strength and fineness satisfactorily, and yet after a time showed signs of disintegration, which led to its rejection for use on our works. On examination by an expert cement manufacturer, the materials of which it was composed proved to be correctly proportioned, and subsequent experience has satisfied me that its defect was due to being lightly burned, and not finely ground. Coarse particles in a cement of this kind hydrate more slowly than the finer particles, and this is followed by expansion and disintegration of neat cement briquettes. If this cement had been ground to a uniform degree of fineness, I have little doubt that it would have proved a thoroughly safe cement. German Portland cement is, as a rule, much more finely ground than English cements, and this is due to the adoption of the mortar test for testing the tensile strength

of cements. The rule adopted there of testing the strength of a mixture of cement and sand, instead of cement alone, has compelled the cement makers to grind their cement very fine. Beyond proving the value of fine grinding, I see no advantages in the sand test over the neat cement; but I do see disadvantages, namely, that it introduces a a second element of uncertainty into our tests. It is all very well to say that we should use a standard sand, but it has been proved, over and over again, that sand obtained even from the same district and of the same grade of fineness will give very variable results. These are some of the reasons why the sand test has not gained ground in England. At the same time, I must freely admit that those who have used the sand test have got the best ground cement. I should much like to hear this point discussed here, for I think now that it is so generally admitted that the finer a cement is ground the better; it is quite possible to get all we require by testing for fineness with very fine sieves, and for strength with neat cement, and thus avoid the secondary issues raised by the introduction of sand into our tests.

Henry Faija, M. Inst. C. E.—I am extremely pleased to find that Mr. Kinipple recognizes the importance of a test that definitely determines whether or no a cement is sound, as it corroborates a theory which I have for many years propounded, viz., that, given a cement which proves itself to be sound and free from either expansion or contraction, it is, in 99 cases out of 100, strong enough for all constructive purposes, and the support of an engineer of Mr. Kinipple's eminence, and who has used more cement than most people, is most gratifying. The test, however, which he proposes is, I think, a little cumbrous and occupies some little time, certainly, except in the case of abnormally bad cements, a few weeks, and I fail therefore to see that it supersedes my own test for the same purpose, which is fully described in the paper. It has now been in very general use for more than 10 years, and enables a perfectly reliable determination being arrived at within 24 hours. A sketch of the apparatus used in this test is given in a paper which I had the honor of communicating to the American Society of Civil Engineers in 1886; but perhaps, in order to bring the matter more forcibly before the notice of cement users, the following sketch (see page 608) will render the description given in the paper more complete.

With respect to the fineness of grinding to which Mr. Kinipple and

Mr. Griffith both refer, I think there are now no two opinions on the matter, as it has been most conclusively proved and established that the finer a cement is ground, the better work it will produce; this result is due to the cement being in greater subdivision and thereby enabled more perfectly to surround each of the pieces of aggregate with which it is mixed, thus giving greater strength or allowing of less cement being used. Such being the case, it is a matter for the manufacturer to say to what degree of fineness he can grind the cement without extra or exorbitant expense, and the economy of fine grinding is therefore defined by the balance between the cost of grinding and the use of more cement; but I would again refer to that portion of my paper in which I pointed out that fineness of grinding, unless the powder is in the form of flour, is very delusive.



It is hardly necessary to refer to Mr. Griffith's remarks about the magnesia scare which obtained here some few years back. I went into the matter very carefully at the time, both theoretically, chemically and practically, and the results of my experiments were com-

municated in two papers to the Society of Engineers in England, and as they brought forth no tangible refutation from those who started the magnesia scare, it may fairly be assumed that my experiments were taken as conclusive, and that the deduction I arrived at, viz., that a good and properly used Portland cement is not detrimentally affected by sea water, is correct.

The absence of a standard specification in this country, which Mr. Griffith laments, has, I think, rather conduced to the production of a better cement than had we been governed by a specification drawn up 15, 10, or even 5 years ago. Had such a specification been in existence there is no doubt that the manufacture in this country would have been to a certain extent limited to compliance with that specification, and would therefore not have improved in the manner which it has. I am quite willing to admit that many of the specifications submitted to manufacturers, and with which they are requested to comply, are often worse than stupid, and, although the manufacturer may be quite unable to comply with them, still they have forced him to do something out of the ordinary, which in the end has resulted in an improved manufacture. While therefore deprecating many of the clauses which are inserted in cement specifications, and while I would myself never condemn a cement because it does not absolutely comply with those terms, I would on no account prevent the layman from writing such specifications as he thinks fit.

Mr. Griffith's remarks about the sand test are quite in accord with my own views, as it is well known that I have from the very commencement fought against the introduction of an extraneous and unknown material into a cement test. Through the universal introduction of the sand test in Germany, the production of cement in that country has resulted in what I should call an adulterated cement, and this is to a certain extent acknowledged by the Germans themselves, as in their own directions to manufacturers it is specially stipulated that no cement shall contain more than a certain percentage of adulterating material. The particular adulterant which is used is sulphate of lime, which not only renders the cement slower setting, but adds somewhat to its adhesive powers. At the same time if this adulterant much exceeds 3 or 4%, it renders the cement an absolutely dangerous one to use, and it seems to me that it is wiser to be satisfied with a good honest Portland cement that perhaps develops a little less tensile strength when

gauged with sand, than one which gives the maximum in this test, but which, by a very slight error in manufacture, may result in the destruction of the whole of the work in which it is used.

I have only to tender my thanks to the Congress for the kind manner in which my paper was received.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

676.

(Vol. XXX.—December, 1893.)

TOPOGRAPHIC SURVEYS.—DISCUSSION ON PAPER No. 635.*

By CHARLES B. BRUSH, KENNETH ALLEN, HENRY GANNETT, MARCUS BAKER and HERBERT G. OGDEN.

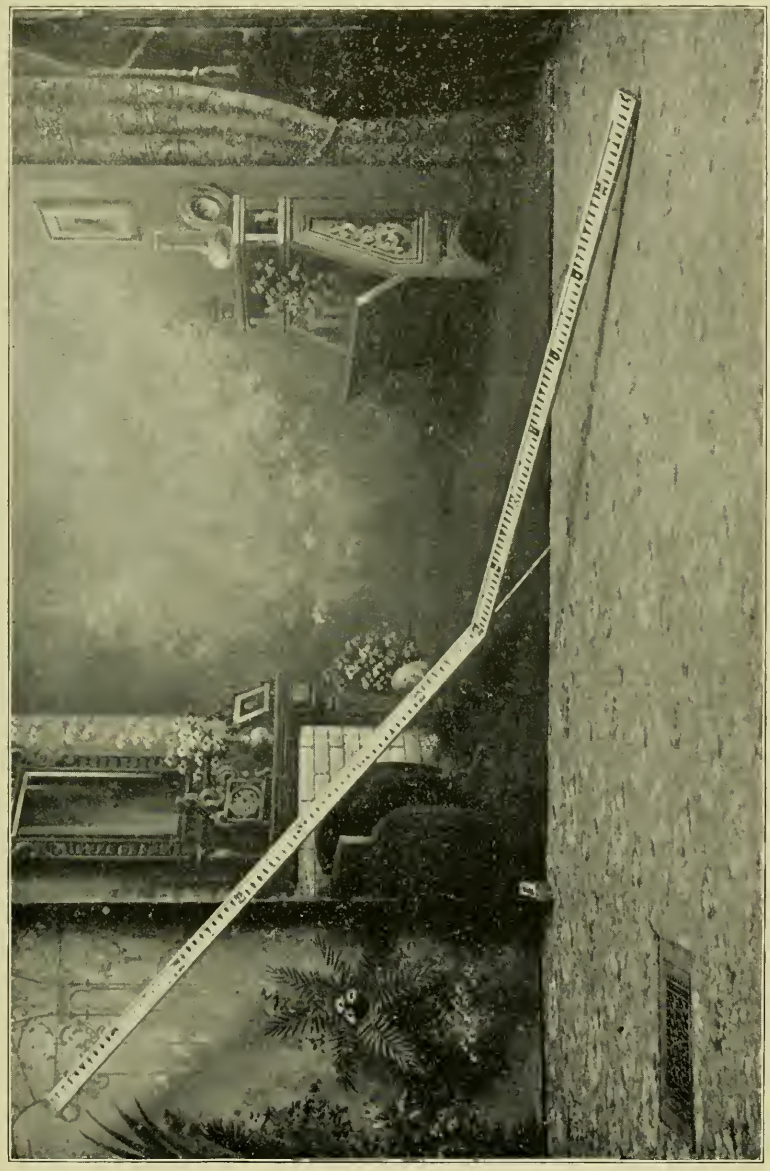
CHARLES B. BRUSH, M. Am. Soc. C. E.—Some of the most interesting exhibits to be seen at this wonderful Exposition at Chicago are the numerous relief maps. I do not know of anything which has interested me more than the relief map of the New York State Canals and of the Nicaragua Canal. They are advanced types of this class of work.

In relation to the topographical maps of the United States, I do not know of any as complete as those of the State of New Jersey. This, I believe, is the only one of our States that has a complete topographical map of its whole area. In New Jersey, these maps have been found to be of great value and are extensively used. Separate sheets of different parts of the State can be purchased to any extent for 25 cents each. These maps were prepared under the direction of Professor Cook, State Geologist.

* "Topographic Surveys." By Herbert G. Ogden, Assistant U. S. Coast and Geodetic Survey. *Transactions*, Vol. XXX, page 62.

KENNETH ALLEN, M. Am. Soc. C. E.—Mr. Ogden's paper is one of great interest in showing the development of topographical surveying in this country. This development has been both in the increased extent of the work done, in field methods and in mapping; and in each of these particulars it has been marked. The surveys of 40 and 50 years ago, valuable as they were at the time, were scarcely more than a compilation of more or less faithful sketches as regarded the actual relief, this being indicated generally by the use of hachures with the occasional elevation of summits and thalwegs. To-day, by the aid of the stadia and vertical arc, the modern range finder, the aneroid and the camera, we are enabled to conduct our topographical surveys with ease and expedition, and hence with economy; and by the introduction of contours of known elevation supplemented only by the occasional use of hachures, our maps indicate the hypsometric features with a precision dependent only on that of the field work and on the scale of the map.

The degree of precision is so dependent on the field methods employed, however, that these, if not indicated in some way on the completed map, should at least be known by those who make use of it. Where such a diversity of methods exists as is quite proper, considering the great difference in topography in various sections of the country and the specific purposes for which surveys are made, it would seem highly desirable that the execution of State surveys should be based upon some such rules or criteria as those formulated by the conference of topographers referred to by Mr. Ogden. For the greater number of surveys where contour intervals of from 5 to 25 ft. are employed, the writer believes, from its use on surveys covering some 600 sq. miles, that there is as yet no more expeditious, economical or flexible method developed than by the employment of the stadia in the ordinary surveyor's transit. It is adapted to every character of country, may be used in parties of from two to six men, depending on the ground and the price of wages, involves but about $10 extra cost in the instrument, and can be used with the ordinary Philadelphia rod, if a special stadia rod is not at hand. In ordinary work in a hilly country, circuits of from 1 to 3 miles in length are run, checking on benches when possible, elevations being determined with the vertical arc. Such circuits, with ordinary care, will check within 1 or 2 ft. in elevation and about 15 ft. in horizontal



distance. With triangulation points not over 3 or 4 miles apart, bench marks established every square mile or so, contours 10 ft. apart and the scale of the map not over $\frac{1}{5000}$, the above error will not be found excessive; while, if greater or less detail and precision are required, the method may to a certain extent be adapted to it with economy.

There are one or two points pertinent to the occasion with regard to instruments. The aneroid is a most useful aid to one who understands its use and its limitations. Many surveying aneroids, as manufactured, carry a vernier on a scale of feet to be read with a glass to single feet. Now, where such accuracy is employed in reading, it is disconcerting to be obliged to make a very appreciable correction for temperature. It would seem more rational to employ the foot scale for very rough work only, and have the vernier on the pressure scale, to be used on more refined work.

The writer has found that with most men, in fair weather and an open country, it is of great advantage to plot the work on a small sketch table mounted on a tripod in the field. On a recent survey of the Connellsville coke region, a number of these were made, having four removable sections over which skeleton sheets containing all known cadastral points were stretched, these four sections being interchangeable in the frame. When the two eastern ones, for instance, were finished, they were shifted to the west side of the table, and fresh sheets were inserted in their places. This avoided folding or overlapping sheets, which had previously been found objectionable. A 14-ft. folding stadia rod was also devised with a graduation which was at once simple, distinct and serviceable. At very long range mirrors inserted at certain points on the rod were easily distinguished, and aided in avoiding mistakes.

In the survey referred to, the area covered was 168 sq. miles, lying in a tract about 44 miles long and in no place over 5 miles wide. For local reasons, traverse circuits were run with steel tape and transit as the base of the survey—in all, 220 miles in length. After plotting on a scale of 600 ft. per inch, the error in a number of these circuits was found to be about 3 in 10 000. The work was done by ordinary chainmen, partly over very hilly ground, and partly along the railroads. Where an error was suspected, the line was re-run.

No angular error in a circuit was allowed to exceed 5′, but

there was obviously no way to control an accumulation of angular error in a tract of this description. Observations for the meridian were taken, however, either on a star or the sun at eight points, averaging about 6 miles apart from one end of the region to the other, and connected with the traverses. The resulting angular errors, after eliminating the effect of divergence of meridians due to the earth's curvature, were in the order of their occurrence as follows: $0'$, $-3'.5$, $+8'$, $+5'$, $+14'$, $-0'.5$, $6'.5$, $-6'.5$. The average angular error in the 52 separate circuits was $2'.05$, the maximum being $5'$.

Mr. Ogden's data as to the cost of topographical surveys is valuable to those States that, following the example of New Jersey, Pennsylvania, Massachusetts, Rhode Island, Connecticut and New York. are agitating the question of inaugurating State topographical surveys, The following figures may be of interest in that connection. On surveys occupying 162 days' field work, made in 1884, for the Philadelphia Water Department, with 10-ft. contour intervals and plotting on a scale of 400 ft. per inch, the party consisting of one transitman and one rodman, the average area covered per day taking notes was 0.434 sq. mile, average area covered per day plotting in the field was 0.310 sq. mile.

	City of St. Louis.	Connellsville Coke Region.	Croton Water-Shed.	Surveys, Philadelphia Water Department.	New Jersey, State Geological Survey.	Massachusetts Topographical Survey.	United States Survey, Southern Appalachian Region.
Area, square miles	44	168	400	446	7 514	8 900	14 200
Contour interval	3	10	20	10	10 & 20	20	100
Scale of map	$\frac{1}{2400}$	$\frac{1}{19200}$[a]	$\frac{1}{56320}$[b]	$\frac{1}{19200}$[c]	$\frac{1}{62500}$	$\frac{1}{62500}$	$\frac{1}{125000}$
Total cost per square mile	$732 80	$116 36	$17 50	.....	$9 00	$13 00	$4 00

[a] — Scale of original map ..... $\frac{1}{7200}$

[b] — " " ..... $\frac{1}{21120}$

[c] — " " ..... $\frac{1}{4800}$

On a survey in the Connellsville coke region for the H. C. Frick Coke Company, Mr. J. H. Paddock, Chief Engineer, which was similar in character to the above, but made with greater accuracy and fullness of detail, and plotted on a scale of 600 ft. to 1 in., the cost in salaries

for field work on topography alone was not far from $40 per square mile. The parties consisted of four men, and plotting was generally done in the field. The total cost per square mile includes, besides the total cost of topographical survey proper, the location of coal out-crops, of all farm boundaries, tabulation of coal areas of each piece of property, observations for meridian, latitude and longitude, and the reduction of all previous surveys of the company to the Scottdale meridian The cost of engraving and publication was in addition $5 275.

The increasing demand for State topographical surveys, as well as for those of less extent, indicates a corresponding appreciation of their value in a variety of fields. The meteorologist and the geologist are aided materially by the data thus made available, and with a reliable contour map at hand the preliminary surveys for railways, canals, water supplies, drainage and irrigation schemes, and for sewerage systems, may be partially or totally avoided, saving, in time, much more than the cost of the survey. It remains for the topographer to select from a multitude of methods and instruments, for the conduct of the survey and the projection and reproduction of results, such as are best adapted to the particular character of the territory with which he has to deal—its extent, the ultimate purpose of the survey, and, last but not least, the means at his disposal to accomplish the work.

Mr. Henry Gannett.*—Mr. Ogden's paper summarizes the views of one of the leading officers of the Coast and Geodetic Survey concerning the topographic map of the United States of the future. It states what, in his opinion, or rather, in the opinion of a conference of topographers of the U. S. Coast and Geodetic Survey, should be the scale, the contour interval, the degree of detail, the maximum allowable errors, and the features which this map should represent. His paper is of special interest to me, as I have from the beginning been in practical control of the survey and construction of the map of the country, which the U. S. Geological Survey is making, and I am pleased to see how closely these ideas and my own are in accord.

Thus Mr. Ogden advocates using a scale of 1:30 000, which, where the features are large and bold, may be reduced to 1:40 000. The Geological Survey is using in the northeastern States a scale of 1:45 000; in the Mississippi Valley and on the plains a scale of 1:31 680, and in the Rocky Mountain region, where the features are large and bold 1:63 360.

* Chief Topographer, U. S. Geological Survey.

It is believed that the last of these scales is amply large for the representation of the essential detail in this bold mountain and desert region. It is to be noted that both Mr. Ogden and myself refer here to the scales of the plane-table sheets, not of the maps as published.

Mr. Ogden advises a contour interval of 20 ft. in the level regions of the country, increasing the contour interval where the relief becomes more pronounced. The U. S. Geological Survey uses, as a rule, the 20-ft. interval. In Florida, where the relief is comparatively slight, it uses a 10-ft. interval, and in Southern Louisiana, where the relief is trifling, a 5-ft. interval is employed. In the Rocky Mountain region the interval ranges from 20 or 25 ft. upon the plains and deserts, up to a 100-ft. interval in the high mountains. Mr. Ogden advises mapping the outlines of wooded areas, as being of importance from many points of view. The U. S. Geological Survey maps these outlines of wooded areas, but does not consider it advisable to publish them upon its larger scale maps, inasmuch as the timber outlines are changing rapidly, but it does propose to publish them upon maps on smaller scales. Mr. Ogden's list of the works of man, or culture, which should be represented, comprises little besides objects which may be denominated public culture; that is, objects which are of interest to the community at large rather than to individuals only. The only exception which I note to this statement is that of houses. The Geological Survey early adopted as its policy in this matter the representation of objects of public culture only. Within recent years it has departed from this practice to the extent of adding houses, and with this addition its list of objects of culture represented agrees very closely with that suggested by Mr. Ogden.

As to the limit of error set by Mr. Ogden, it may be asked how the errors of the map are to be determined absolutely, since any work done to test the map is itself liable to error. Thus the stipulation that the height of a hill shall be within 5 ft. of the true height implies a knowledge of the true height of the hill, but how is this to be obtained. It may be suggested that verification of a map can be made by a duplication of the work and a comparison of the results, but this will show, not the errors of the original map, but the discrepancies between the two maps, either of which may be in error.

We can estimate roughly the value of the computed triangulation by the closures of the triangles. We know that graphic triangulation

carried on by the plane table proves itself, since, if sensible errors are introduced, the work falls to pieces. We can detect errors of magnitude in traverses made by the plane table, by their closures upon triangulation points, and we can detect errors in sketching by a comparison of the map with the ground. But since we have nowhere anything absolute to compare with, I do not see how we can measure the amount of errors of location, and say that the position of an object is in error to a greater or a less extent than that admissible, except where the errors are gross.

Now as to cost. The cost of the topographic work of the U. S. Coast and Geodetic Survey at present, including the triangulation upon which it rests, is about $200 per square mile. It is not to be understood that the work in the interior would cost as much, since the scale which Mr. Ogden proposes is much smaller. He estimates its cost at $35 to $45 per square mile, or, say, an average of $40 per square mile. At this rate, the survey of the whole United States, excluding Alaska, would cost $120 000 000. It is not at all probable that the United States would authorize so expensive a piece of work to be undertaken.

On the other hand, the work of the Geological Survey, which is pursued practically upon the plan advocated by Mr. Ogden, has cost for the past three years an average of $5 per square mile, the cost ranging from $3 upon the western plains to $10 in New England and New York. The total cost of the work, one-fifth of the country having been completed already, will not exceed $15 000 000, being but a small fraction of the sum above estimated.

This great difference in cost requires an explanation. A part of it can be very simply accounted for in the difference in policy governing the work of the two organizations. The policy of the U. S. Coast and Geodetic Survey has been throughout to attain the highest possible degree of accuracy. The Geological Survey, on the other hand, aims only at such accuracy as is required for the specific purpose in hand, as the engineer in constructing a bridge apportions the strength of each member to the strain it is to bear.

In pursuance of its policy, the U. S. Coast and Geodetic Survey locates by triangulation and computes the positions of a large number of points, many times the number required for instituting plane table-work. Moreover, it spends time and money in attaining a degree of

accuracy in the location of these points many times greater than is required for the map. The U. S. Geological Survey, on the other hand, locates by triangulation and computes the position of a minimum number of points, three to each plane table sheet, relegating the work of location thereafter to graphic methods with the plane table, which are, of course, much cheaper and of sensible accuracy. Moreover, the location of points in its primary triangulation is only of such accuracy as is required by the scale of the map, since errors which are not sufficiently large to show upon the map are not errors in a map sense. This proportioning of means to ends reduces the cost of the work of the U. S. Geological Survey by nearly one-half, since triangulation, such as is executed by the U. S. Coast and Geodetic Survey, costs, according to their own statement, very nearly as much as the topographic work proper, while the triangulation carried on by the U. S. Geological Survey probably costs not more than one-tenth as much as the topographic work.

But this does not account for the entire difference between the cost of the work of the U. S. Geological Survey and Mr. Ogden's estimate, and I cannot help feeling that his estimate is needlessly high, even for work which may be done by the organization which he represents. The figures given for the U. S. Geological Survey are not estimates. They are the results of experience covering a term of years and hundreds of thousands of square miles, and certainly are entitled to greater credence than any estimates.

There is another item to be considered besides expense in this matter, and that is time. An expenditure of $40 per square mile implies slow progress or a very large annual appropriation. At present the U. S. Geological Survey is spending about $250 000 annually in topographic work, at an average cost of $5 per square mile. Its average annual output is about 50 000 sq. miles. To survey this area at a cost of $40 per square mile would require an annual appropriation of $2 000 000. It is not at all probable that Congress would support it to this extent. The alternative, therefore, is slow progress. Now, if maps are of value to the industries of the country, and we all agree that they are, that value is enhanced by the rapidity of the work. The sooner we have maps, the more use will they be to our industries. Roads, railroads, irrigation works, aqueducts and other improvements are being constructed all the time, and the more of

them which have the benefit of maps, the greater is the utility of those maps. The more rapidly this work can be pushed forward to completion, other things being equal, the more valuable it will be, and, therefore, in considering plans for this work, the item, not only of expense, but of time, should receive due consideration.

Under an annual appropriation of $250 000, the U. S. Geological Survey maps 50 000 sq. miles, and will finish the map of the country, excluding Alaska, in 50 years. This, it would seem, is sufficiently long to wait for it. But under Mr. Ogden's estimate, with the same appropriation, 6 250 sq. miles would be the annual output, and the entire map would be finished in the year 2373, 480 years from the present date.

Mr. Marcus Baker.—Some months since I compiled from published sources some memoranda respecting the cost of making topographic surveys and maps. Such data in compact form, it was believed, would be useful for reference. It seems the more important to do this because the question of cost has been to some extent a matter of controversy.

The subject is one of great practical importance, and it seems to me that the statements of cost given by Mr. Ogden may profitably be extended. This opportunity is therefore taken to offer these memoranda for publication as a contribution to, or extension of, that part of Mr. Ogden's interesting and valuable paper relating to the cost of topographic surveys.

Foreign Surveys.*

India.—It is not easy to get figures showing cost of foreign surveys that are of much value for comparison with similar figures for the United States. Diversity of conditions, of country surveyed, of scales, of purposes for which the maps are made, etc., greatly affect cost.

The country whose conditions appear to be most nearly like our own, and whose experience, therefore, is of most value to us, is India.

In India 10 years' experience in making maps on the 1 in. to 1 mile scale gives a cost of $11 per square mile, and on the 4 ins. to 1 mile scale a cost of $26.50. A similar 15 years' experience in cadastral mapping on the 16 ins. to 1 mile scale gives a cost of $82 per square mile.

* Most of the data for foreign surveys are compiled from Captain George M. Wheeler's Report upon the Third International Geographical Congress and Exhibition at Venice, 1881. 4to, Washington, 1885.

In the Madras presidency 65 935 sq. miles were mapped on scales of 16, 4, 2 and 1 ins. to 1 mile for a total cost of £71 824, or about $5 50 per square mile.

The town and island of Bombay were surveyed between 1865 and 1872 at a cost of £31 306 = $152 000, the area being 22.23 sq. miles, or at an average cost of $6 815 per square mile.

The ratio of scale to cost in the Indian work is estimated by General Walker, chief of the Survey, to be 16 ins. to 1 mile, 7½; 4 ins. to 1 mile, 4; 1 in. to 1 mile, 1; *i. e.*, the 4 ins. to 1 mile map costs four times as much as the 1 in. to 1 mile map. Similarly, the 16 ins. to 1 mile map costs seven and one-half times as much as the 1 in. to 1 mile map.

Austria-Hungary.—Area, 241 550 sq. miles. An entirely new survey of the monarchy was ordered in 1872. Scale adopted, 1 : 75 000; number of sheets required, 720. Original plane table sheets are on a scale of 1 : 25 000. Relief expressed by hachures, contours and figures. No good estimate of the cost of this map is available. Captain George M. Wheeler, in his report upon the Third International Geographical Congress at Venice, from which most of these statements relative to foreign surveys have been taken, says, p. 275: "An estimate was obtained of the cost of obtaining the field notes (1 : 25 000) for the new special map at approximately $400 per square mile."

Baden.—The area of Baden is 5 820 sq. miles, and since 1875 a detailed topographic survey, scale 1 : 25 000, has been in progress. The resulting contour map is to consist of 170 sheets. The cost of the work is stated to be—

For field and office work	$11 90	per	square mile.
For engraving and printing....	10 30	"	"
Total	$22 20	"	"

Bavaria.—The area of Bavaria is 29 280 sq. miles. A cadastral survey (no elevations shown) on a scale of 1 : 5 000, begun in 1808, was finished in 1860. The cost of this survey is estimated at $79 per square mile.

For many years a topographic map has been in progress. The surveys for this consist of a revision of the earlier cadastral map with the addition of elevations and drawing of contours. The amount expended for the work of the Topographic Bureau from the beginning of

its work down to 1885 is estimated at $1 750 000, or at the rate of about $60 per square mile for the whole country.

The current field cost of the altitude determinations, topography and revision, is stated to be from $9 to $13 per square mile.

Belgium.—Area, 11 375 sq. miles. All mapped on a scale of 1 : 20 000, and 1 : 40 000; contour interval, 1 m. Has been undergoing a complete revision. This revision has cost about $23 per square mile. An estimate made a few years since gives $176 as the cost per square mile of the Belgian surveys. The cost of engraving is stated to be about $6 50 per square inch.

France.—The director of the survey states the cost of the surveys of precision by the engineer brigade (scales 1 : 10 000 and 1 : 20 000) to be 100 francs per square kilometer, or about $52 per square mile.

The cost of the survey of Algeria, scale 1 : 50 000, is, for field work, 20 francs per square kilometer, and for engraving, printing, etc., 40 francs per square kilometer, a total cost of about $31 per square mile.

Great Britain.—Area of the United Kingdom, 123 180 sq. miles. Total expenditures for the Ordnance Survey from its creation in the last century down to 1881 was $22 725 000, giving an average expenditure for surveys of $184 per square mile.

The cadastral survey of the United Kingdom, scale of map 1:2 500, began in 1862, and has cost an average of 23d. per acre, equal to $294 per square mile.

Italy.—In 1862 Italy began the preparation of a general topographic map of the entire kingdom on a scale of 1 : 100 000, the map to consist of 277 sheets. The original field maps are on a scale of 1 : 25 000 and 1 : 50 000, with contours 5 and 10 m. apart, respectively. The area of the kingdom is 111 400 sq. miles, and the cost is estimated at from $3 300 000 to $5 000 000, making a cost of from $30 to $45 per square mile.

The director of the survey in 1885 estimates the cost of a single sheet, area 625 sq. miles, of the 1 : 100 000 map at $1 000 for triangulation, and $2 400 for the topography, if done on the 1 : 50 000 scale, but $7 200 if done on the 1 : 25 000 scale, or at a cost of from $5 44 to $13 12 per square mile.

Prussia.—Area, 159 950 square miles. A topographic map is now in progress on a scale of 1 : 100 000. The original sheets are on a scale of 1 : 25 000, with contour intervals of 5 m. (about 16 ft.). The pres-

ent rate of cost for this map is, for triangulation, $22 45; for topography, $28 35, and for drawing, engraving and publishing, $20 10 per square mile, a total cost of about $71 per square mile.

Russia.—Of the topographic work now in progress, and which has been in progress since 1870, the director estimates the cost at 29.5 rubles, or about $22 per square mile.

Cost of Topographic Surveys in the United States.

United States Geological Survey.—The Geological Survey began systematically to make a contour topographic map of the United States in 1882. It adopted three scales, 1 : 250 000, or about 4 miles to 1 in.; 1 : 125 000, or about 2 miles to 1 in., and 1 :62 500, or about 1 mile to 1 in. The contour interval has varied from 5 to 250 ft. according to scale of map and character of country to be mapped. Early in the history of the work much was done on the smallest scale and little on the largest scale. At present and for a few years past the smallest scale work has been abandoned, and a steadily increasing amount of work has been done on the largest scale.

The total area mapped from the beginning to June 30th, 1893, is 583 922 sq. miles, and the total cost for all expenditures whatsoever necessary to produce the finished manuscript map ready for the engraver is $2 496 776, an average of $4 27 per square mile.

The items of cost include all salaries, instruments, repairs, supplies, superintendence, disbursement for, not only the topography proper, but for all astronomical work, base measurement, triangulation and leveling, etc., which are auxiliary thereto.

The areas mapped on the different scales are approximately as follows:

1 : 62 500	80 000 sq. miles.
1 : 125 000	300 000 "
1 : 250 000	200 000 "
Total	580 000 "

The following table shows the cost by years from the beginning to the end of June, 1893. The first column gives the fiscal year; the second, the total appropriation for the survey; the third, the total appropriation for topography; the fourth, the percentage of the ap-

propriation for topography to the whole appropriation; the fifth, the area mapped, in square miles; and the sixth, the cost per square mile.

UNITED STATES GEOLOGICAL SURVEY.

Cost of Topography.

Fiscal Year, ending June 30th.	Appropriation.	Topography.	Percentage of appropriation.	Area. Square miles.	Cost per square mile.
1883	$256 940	$70 700	27	15 000	$4 71
1884	339 640	88 600	26	45 150	1 96
1885	489 040	160 000	33	57 500	2 78
1886	503 240	184 510	37	81 829	2 25
1887	503 240	204 656	41	55 684	3 67
1888	503 240	202 310	40	52 062	3 88
1889	605 240	278 200	46	64 472	4 32
1890	494 657	336 200	68	67 650	4 99
1891	618 615	361 200	58	65 575	5 51
1892	589 600	266 200	45	53 000	5 02
1893	376 100	249 200	66	26 000	9 58
All	$5 279 552	$2 401 776	45	583 922	
From New Jersey (say)	.....	25 000			
Massachusetts	.....	40 000			
Rhode Island	.....	5 000			
Connecticut	.....	25 000			
		$2 496 776		583 922	$4 27

This average cost is derived from work on three different scales. As the scale affects the cost, the following specific cases—New Jersey, Massachusetts, Rhode Island and Connecticut—of work executed under the direction of the U. S. Geological Survey, and all on the 1 in. to 1 mile scale will afford useful data in determining the cost of that class of work in the thickly settled eastern part of the United States.

New Jersey.—The topographic survey of New Jersey was begun in 1877 and completed in 1887. The total area of the State is 7 894 sq. miles, and the cost of making the contour topographic map designed to be as accurate as the scale of 1 in. to 1 mile will admit of, was $54 744, being at an average cost of $6 93 per square mile.* Of the total cost, about one-half was borne by the State, and the remainder by the Coast and Geodetic Survey and by the U. S. Geological Survey. This cost covers all expenses whatsoever incurred in the production of the manuscript map ready for the engraver. The scale of the drawing was 3 ins. to 1 mile; contour interval, 10 and 20 ft. In his final

* Geological Survey of New Jersey, Annual Report, 1887, page 15.

report, Dr. Cook, State Geologist of New Jersey, estimates that the cost would have been not to exceed $10 per square mile in case the State had been required to execute its own triangulation for the control of the map.

Massachusetts.—The State of Massachusetts was mapped by the U. S. Geological Survey, in co-operation with the State, in the years 1884 to 1888. The resulting map, on a scale of 1 mile to 1 in., and with 20-ft. contours, contained on 53 atlas sheets, cost, for all expenses of every kind and character necessary to the production of the manuscript map ready for the engraver, $107 800. Of this amount, $40 000 was paid by Massachusetts, and the remainder by the U. S. Geological Survey. The average cost of the Massachusetts survey was $12 97 per square mile.

Rhode Island.—The State of Rhode Island was mapped, in the years 1888 and 1889, by the U. S. Geological Survey, in co-operation with the State. The map produced, composed of 15 sheets, similar to those in Massachusetts, cost $9 732 51, one-half of which was paid by Rhode Island and one-half by the U. S. Geological Survey. Taking the area of the State to be 1 085 sq. miles, the average cost was $8 97 per square mile.

Connecticut.—The State of Connecticut was mapped by the U. S. Geological Survey, in conjunction with the State, during the years 1889 and 1890. The total cost of the survey for expenses of every kind and character, such as salaries, traveling expenses, subsistence, pay of men in office engaged in drawing the maps, etc., including everything necessary to the production of the manuscript map ready for the engraver, was $48 999. One-half of this cost was paid by Connecticut and one-half by the U. S. Geological Survey. The area of the State being 5 004 sq. miles, this gives an average cost per square mile of $9 79. This work, done under the direction of the U. S. Geological Survey, yielded a manuscript map of the State composed of 33 atlas sheets, all on a scale of 1 mile to 1 in. and with 20-ft. contour intervals.

Perkiomen Water Basin.—A survey of this basin, for increasing the water supply of the city of Philadelphia, was made by Rudolph Hering, M. Am. Soc. C. E., under direction of Colonel William Ludlow, M. Am. Soc. C. E., and completed in 1885.

The survey comprises an area of 446 sq. miles in Bucks, Montgomery, and Lehigh Counties, Pa. Scale of the map, 400 ft. to 1 in.,

or 1 : 4 800. It has 10-ft. contours and shows all roads, buildings and timber outlines.

I am informed by Mr. Henry Gannett that the cost of this work was stated by Mr. Hering to be $145 per square mile.

Mississippi River Commission.—A topographic map of the banks of the Mississippi River from Cairo to the Gulf of Mexico was completed by the Mississippi River Commission in 1884. The manuscript maps on a scale of 1:10 000 embrace the river and a strip of land on either bank about ½ mile in width. The river was carefully sounded, and the resulting map of the river and its shores represents an area of 1 954 sq. miles of land and water surface, of which the cost was $99 930, or $51 per square mile.*

Lake Survey.—A survey of the northern and northwestern lakes was made between 1841 and 1881, by the Corps of Engineers, U. S. A. The total sum expended on the work was nearly $3 000 000.

The shore line was accurately surveyed and topographic maps made of a strip along the shore about ½ mile wide. This work was based on a careful triangulation, was done on a scale of 1:10 000, and cost for the topography and the triangulation executed for topographic purposes somewhat more than $125 per square mile. The triangulation executed for control of a long and narrow strip of topography is available for a wider strip. If, however, a wide strip is not surveyed, the cost of the area is increased. The cost of $120 per square mile is the amount estimated by General Humphrey and General Comstock as the cost of accurate topography on a scale of 1:10 000, when mapped in considerable bodies. The cost of the topography of the Lake Survey in 1872 and 1873 was $326 per square mile, but this was exceptional, owing to a relatively large amount of triangulation and small amount of topography executed in those years.

Coast Survey.—The Coast Survey created in 1807 did its first surveying in 1817. It was interrupted the following year and resumed in 1832, since which date it has been carried on without interruption. The cost of the topography executed by it has been the subject of controversy, and several estimates have been made. These estimates differ widely. Some of them are here given.

---

* Report of the Mississippi River Commission for 1884, being Appendix W W of Report of the Secretary of War. H. of R. Ex. Doc. 1, Part 2, 49th Congress, 1st Session. Vol. 2, Part 4, p. 2574.

Professor Bache, second superintendent of the survey, in 1848 estimated* that the topography proper executed in 1847 had cost $13 76 per square mile; but if the base measures, triangulation, reconnaissances, astronomical and magnetic work be included, then the cost was $27 71 per square mile.

General C. B. Comstock in 1875, from data published in the Coast Survey reports for the five years 1864–68, estimated† that the triangulation had cost $343 per square mile, and the topography $241 per square mile, making a total cost for each square mile of land surface topographically mapped, $584.

This estimate was warmly repudiated by Mr. C. P. Patterson, then superintendent of the Coast Survey, who declared‡ that the most expensive piece of work executed by the Coast Survey for many years was that of Mt. Desert, Me., and that its cost for all expenses whatsoever was $97 per square mile, while other areas had cost only $23, and yet others only $12, per square mile.

Thereupon General Comstock made a new estimate§ of the cost of the topographic work executed by the Coast Survey, from data published by the Coast Survey for the 14 years 1858–72. This new estimate, based on 14 years' experience, gives as the average cost per square mile of the areas within which the topography and triangulation were completed and the map published, as $403. This result is obtained by charging the whole appropriation to three classes of work, viz., triangulation, topography and hydrography, the cost of the three classes being $120 for triangulation, $283 for topography, and $80 for hydrography. The total cost of the topography, including the triangulation by which it is controlled, is therefore on this estimate $403 per square mile.

Captain George M. Wheeler in 1886, made an estimate‖ of the cost of the three great classes of Coast Survey work, viz., triangulation, topography and hydrography, based on its entire experience, from the beginning in 1807 to 1886, and concludes that the 32 000 sq. miles then topographically surveyed had cost $333 per square mile, $25 being chargeable to triangulation, and $308 to topography proper.

* Coast Survey Report, 1848, pp. 3–4.

† Sen. Ex. Doc. 21, 45th Congress, 3d Session, p. 12.

‡ H. of R. Ex. Doc. 29, 45th Congress, 3d Session, p. 2.

§ Sen. Ex. Doc. No. 21, Part 2, 45th Congress, 3d Session, p. 2.

‖ Wheeler (G. M.), Report on Third Geographical Congress at Venice. 4to, Washington, 1885, p. 533 *et seq.*

The Hon. Hilary A. Herbert, after carefully reviewing the foregoing estimates, gives a figure of $250 per square mile* as an estimate not unfair to the Coast Survey.

To this list of estimates I will add one more. And this estimate is designed to get at a fair evaluation of the cost of the topographic work now in progress and such as has been done in the past few years, rather than to get at the cost from the beginning.

The Coast Survey report for 1890, the latest one received, contains a detailed and classified statement of expenditures, and thus affords the data for determining the cost of various classes of work for that year.

From it we have, neglecting odd dollars and cents—

Party expenses, 1889–90	$156 290
Permanent salaries	235 500
All other expenses	116 580
Total expenditures	$508 670†

In this same report the party expenses are classified under thirteen heads, which, arranged in groups, are as follows:

Hydrography	$51 878
Physical hydrography	6 510
Tidal operations	5 111
Coast pilot	4 500
Magnetics	3 900
	$71 899
Topography	$25 032
Triangulation in immediate connection therewith	23 757
	$48 789
Transcontinental geodetic triangulation	$21 032
Triangulation for States	7 175
Precise leveling	3 040
Astronomical work (longitudes)	2 646
Gravity experiments	1 700
	$35 593

* H. of R. Report 2740, 49th Congress, 1st Session, p. 77.

† This figure is from the Digest of Appropriations for 1889–90. The Coast Survey report for 1890 gives as the appropriation $530 458 13; and as the expenditure, $485 992 76, leaving an unexpended balance on hand of $44 465 37. If this balance was turned back into the Treasury, the above sum should be diminished by $22 667; if it was all used, the sum should be increased by $21 788.

The first group contains the items chargeable to hydrography exclusively. The item magnetics is a small one, and this work being undertaken primarily for the mariner, it may properly be charged to hydrography.

The second group contains the items topography proper and triangulation, upon which it more immediately depends, *i. e.*, the secondary and tertiary triangulation. It does not include the primary triangulation nor the astronomical and base-measuring work upon which it depends, these latter being charged to the third group.

The third group is made up of the geodetic elements of the Survey, 79% of it being for triangulation. All of these items of expenditure, except the gravity experiments, are incurred for the purpose primarily of making accurate maps, topographic and cadastral, and secondly, or incidentally, of determining more accurately than we now know the size and shape of the earth. For these purposes the larger part of the cost is properly chargeable to topography.

The whole expenditure of $156 290 during the year 1889–90, called "party expenses," may therefore properly be charged against three classes of work, as follows:

Hydrography	46%	$71 900
Topography and the triangulation	31%	48 790
Geodetic work (including triangulation)	23%	35 600
Total party expenses		$156 290

This division of party expenditures we shall now assume to be a fair basis upon which to apportion the parts of the permanent salary list and miscellaneous expenses chargeable to the three groups of hydrography, topography and geodesy, and so doing, have this table:

	Party Expenses.	Percent.	Salaries.	Miscellaneous Expenses.	Total.	Area. Square miles.	Cost per square mile.
Hydrography...	$71 900	46	$108 330	$53 765	$233 995	5 314	$44
Topography....	48 790	31	73 005	36 233	158 028	839	188
Geodesy.....	35 600	23	54 165	26 882	116 647	.....	....
Total	$156 290	100	$235 500	$116 880	$508 670	.....	....

The cost of the topographic work executed in 1889-90 thus comes out $188 per square mile, which amount also includes the cost of engraving. This amount is probably somewhat too low, since a part of the cost of geodetic work should be charged to topography, and also because by reason of the service rendered by naval officers to hydrography, less than 46% of the salary list should be charged to hydrography.

These results are based on the single year 1889–90. They do not, however, differ much from those similarly derived for several years back.

The party expenses of the past seven years are shown in the following table. The classification is the same as that set forth for 1889–90. The percentage of expense chargeable to three great classes of work, viz., topography (including the smaller triangulation incident thereto), hydrography and geodesy, are $\frac{1}{3}$, $\frac{2}{5}$ and $\frac{1}{4}$.

PARTY EXPENSES (PERMANENT SALARIES NOT INCLUDED) OF THE UNITED STATES COAST AND GEODETIC SURVEY.

Fiscal years ending June 3d.	Triangulation and Topography.	Hydrography.	Geodesy.	Total.
1885	$66 797	$56 286	$40 953	$164 036
1886	67 768	72 240	45 334	185 342
1887	54 005	57 564	32 266	143 835
1888	57 070	64 569	35 489	157 128
1889	51 996	72 102	33 353	157 451
1890	48 789	71 909	35 592	156 290
1891	40 982	76 006	43 773	160 761
Sum	$387 407	$470 676	$266 760	$1 124 843
Mean	$55 344	$67 239	$38 108	$160 692
	34%	42%	24%	

These percentages afford, I think, the best means of judging the cost of the different classes of work in the past few years.

The work classed as geodetic is in the main made up of the two items, "Transcontinental Geodetic Triangulation," and "Furnishing Points to States," those items constituting about 70% of the whole. There is included precise leveling and telegraphic longitudes. As all these operations are primarily designed to form the foundation of a map of the whole country, and only secondarily and incidentally to

contribute to a better knowledge of the size and shape of the earth, this portion of the expenditure is properly chargeable to the cost of the map which is to be made. It should not, however, be charged to the cost of the maps into whose contribution it does not enter.

The average expenditure of the Coast Survey in the last eight years has been $509 000, and the average area topographically mapped has been 800 sq. miles per year. Of this amount expended, 34%, or one-third, has been used for the topography and secondary and tertiary triangulation connected with it. This yields an average cost of $212 per square mile.

This cost includes the engraving, but does not include any expenditure for the base measurements, astronomical work or primary triangulation. I regard an estimate of $200 per square mile as rather below than above the truth, and as entirely fair to the Coast Survey.

The estimates of the cost of the topographic work executed by the Coast Survey may therefore be summarized as follows:

Period.	Cost per square mile of—			Estimate made by—
	Triangulation.	Topography.	Total.	
1847.....	$13 95	$13 76	$27 71	Alex. D. Bache, Supt. C. S.
1864–68.....	343 00	241 00	584 00	Gen. C. B. Comstock, U. S. Engs.
.....	.....	12 to 97	.....	C. P. Patterson, Supt. C. S.
1858–72.....	120 00	283 00	403 00	Gen. C. B. Comstock, U. S. Engs.
1807–86.....	25 00	308 00	333 00	Capt. Geo. M. Wheeler, U. S. Engs.
.....	.....	.....	250 00	Hon. Hilary A. Herbert.
Recent years..	.....	.....	200 00	Marcus Baker.

Map of the District of Columbia.—A topographic map of the District of Columbia was begun in 1881 by the Coast Survey in conjunction with the Engineer Office of the District of Columbia. The scale adopted was 1:4800, or about 13 ins. to 1 mile, and the contour interval, 5 ft. The party expenses were specifically provided for in the District of Columbia Appropriation Bill, which for the 11 years, 1881 to 1891, inclusive, appropriated annually an average sum of a little less than $6 000.

The land area of the District of Columbia is 60 sq. miles, of which 11.8 sq. miles lie within the original limits of Washington and Georgetown, and were not included in the survey. Hence, the survey covered 48.2 sq. miles, and for this area the party expenses amount to $65 600, or $1 361 per square mile.

In the experience of the Geological Survey, party expenses constitute one-third of the cost of the finished manuscript map. There is little departure from this ratio in a rather extensive experience. If the same ratio applies in Coast Survey experience, then the cost is $4 083 per square mile, or $6 40 per acre. Assuming, however, that salaries of the officers and surveyors who did this work were only just equal to the party expenses, the cost would be $2 722 per square mile. The data for a close estimate are not at hand, but I am of opinion that the cost of the manuscript map of the survey will be found not less than $3 000 per square mile, or nearly $5 per acre.

The appropriations for this work, made by Congress, 1881–91, were "for surveys of the District of Columbia with reference to the future extension of various avenues to the district line," and are as follows:

Fiscal year ending June 30th.	Appropriation.	Reference.
1881	$5 000	United States Statutes at Large, Vol. 21, p. 156.
1882	5 000	" " " " 21, p. 458.
1883	5 000	" " " " 22, p. 135.
1884	5 000	" " " " 22, p. 464.
1885	5 000	" " " " 23, p. 125
1886	5 000	" " " " 23, p. 313.
1887	4 000	" " " " 24, p. 132.
1888	4 000	" " " " 24, p. 573.
1889	10 000	" " " " 25, p. 319.
1890	10 000	" " " " 25, p. 799.
1891	7 600	" " " " 26, p. 298.
	$65 660	

SUMMARY OF THE COST OF TOPOGRAPHIC SURVEYS IN THE UNITED STATES.

	Area. Square miles.	Cost.	Cost per square mile.	Scale of manuscript map.
United States Geological Survey	583 922	$2 496 776	$4 27	1 : 125 000 1 : 62 500 1 : 45 000 1 : 30 000
New Jersey	7 894	54 744	6 93	1 : 21 120
Massachusetts	8 315	107 800	12 97	1 : 30 000 and 1 : 45 000
Rhode Island	1 085	9 733	8 97	1 : 45 000
Connecticut	5 004	48 999	9 79	1 : 45 000
Perkiomen Water Basin	446	....	145	1 : 4 800
Mississippi River Commission	1 954	99 930	51	1 : 10 000
United States Lake Survey	.....	.....	125	1 : 10 000
United States Coast Survey	.....	.....	200*	1 : 10 000 and 1:20 000
District of Columbia	48.2	.....	3 000	1 : 4 800

* Publication included.

Mr. Herbert G. Ogden.—Mr. Allen's remarks are a valuable contribution to this subject, in giving us some definite data on the cost of several surveys for special purposes, and especially in the details of the errors of closure in work of this kind ; information, it seems, very difficult to obtain. Such information bears directly on the recommendation that some rules, or a criterion, should be established, by which the results of topographic surveys should be gauged, and I am exceedingly pleased that he approves of the suggestion. I wish that he might go further, and, indeed, that all civil engineers would take an interest in the subject, and express themselves personally and through their societies on this important question of a criterion, particularly in the details, specifying the degree of accuracy that should be required to render general topographic work, that could be obtained within a reasonable cost, of the best quality for engineering purposes.

Mr. Gannett's statements would have been of more value to us if he had specified the degree of accuracy that is obtained in the work conducted under his direction. We may all be quite agreed that certain features should be represented ; but how accurately they should bear their true relations to each other has often been in dispute and still seems far from settlement. It is far from satisfactory that Mr. Gannett should ask "how the errors of a map are to be determined absolutely, since any work done to test the map is itself liable to error." Indeed, this would imply that the degree of accuracy attained in his own work has not yet been fully ascertained. That "two surveys of the same ground will only serve to illustrate the differences between the surveys," is a statement that will hardly be accepted by surveyors. Mr. Allen has shown us how work under his cognizance has been checked, and I am quite sure, as he suggests, that any statement on the face of a map that will explain the methods that were employed in making the survey would convey to the great majority of engineers the degree of accuracy in the completed work, provided, of course, they have confidence in the fidelity of the surveyor who executed it. But a re-survey is never necessary to determine the accuracy of a piece of work ; an instrumental examination can always be made, covering large areas, in a very short period, that will develop the general reliability of the map. No more is it necessary that we should know the height of a hill to be sure that our survey has determined it within 5 ft., for experience has demonstrated that certain

instruments carefully used will produce certain results ; and in all surveys there are more or less opportunities to bring checks on elevations, as well as distances, that we know from experience must determine the results within well-defined limits. Mr. Allen's example is a very happy one, but will any of us believe that if his distances had been measured by an odometer, his directions by a compass, and elevations by an aneroid barometer—the method followed in much of the traverse work of the Geological Survey*—that his work would have checked as he has represented ? Mr. Gannett admits that checks are available in some descriptions of work ; we have only to specify what discrepancies should be permitted in using these checks for distances, and establish like rules for elevations, to form a reasonable criterion.

I have already stated that it is impracticable to prepare an estimate of the average cost of the topographic work of the Coast and Geodetic Survey, although many attempts have been made to do so. The figures presented by Mr. Gannett, about $200 per square mile, are doubtless derived from the estimates recently prepared by his associate, Mr. Marcus Baker. Estimates of a similar character were presented by General Comstock some years ago, placing the average cost at something over $400 per square mile. More recently, Captain Wheeler found $330 more nearly correct ; and now Mr. Gannett presents revised figures, reducing the sum to $200. I sincerely trust our friends who are interested in this matter will continue their figuring in the hope that eventually it will bring the price down to something that can at least be admitted as an approximation. We know very well the cost of many individual pieces of work ; those to which I have referred represent the cost of areas that have been among the most difficult and expensive; and an estimate, therefore, of the average cost of the topography of the Coast Survey that is more than twice as great as these individual instances cannot have much reliance placed upon it.

These excessive estimates for the cost of Coast Survey topography are largely due to the methods employed in preparing them, much more to this cause than to the details of the operations Mr. Gannett has referred to, though doubtless the production of accurate work is more expensive than work of a lower grade. They have uniformly

* Report of Superintendent, Coast and Geodetic Survey, 1891, Appendix 16, page 708.

been based upon an assumption that in the total expenditures for the Survey the cost of triangulation, topography and hydrography must have been incurred in fixed ratios to the whole amount, and that all the expenditures for the Survey are properly chargeable to one of these classes of work. This assumption ignores the fact that large sums are expended annually in the maintenance of published work, the publication of results, support of plant, executive work, special surveys, re-surveys and examinations, and similar operations that do no pertain to the three principal classes, but, having been included in these estimates, render the ratios worthless. To put figures derived from such sources beside those from the Geological Survey, which embrace only the completion of the manuscript map, or beside those for the State surveys of Massachusetts or New Jersey, which go no further, and also omit the cost of triangulation, is calculated to mislead and would vitiate any estimates that should be based upon them.

The field work of the topography of the Coast Survey is so involved with expenditures for triangulation and hydrography, executed by the same parties and paid for on the same rolls, that it is impossible to separate the charge for topography from other expenditures; but even if we could do so, the cost of the narrow strip of work embraced in our operations would not be a guide, and could hardly be used in estimating the cost of topographic work away from the coast, since the details of shore-line, islands, ledges, rocks and low water that are represented on our sheets, besides the difficulty and great expense incident to maintaining field parties on many sections of the coast, necessitate expenditures greatly enhancing the cost and that would not be incurred on plain interior work.

Mr. Gannett's estimates for the time required to survey the whole United States are unique, and might be used for an argument if they were based upon correct premises. Unfortunately for the argument, however, he has interpreted my statement that the cost of topographic work would not exceed $35 or $45 per square mile for the most difficult and detailed areas as an estimate for the average cost per square mile. I regret that he should have so misunderstood my plain statement, as it involves him in a computation that runs forward to the year 2373 A. D. In point of fact, the suggestions I made were to accomplish the survey in a generation, which is even a shorter time than is now claimed would be necessary to complete the topographic

work of the Geological Survey. Joint work by the National Government and the States would in a very few years permit a moderate assistance to all the States, and certainly, if all the States were engaged at once in making surveys within their own territories, we would hardly anticipate a delay in the final results of 480 years, or even 50 years. But all estimates predicting the time within which such a great work can be accomplished are tentative and subject to modification by changes occurring in the future, and over which we have no control; and I look, therefore, upon estimates of this character as very largely a waste of time. In 1856 the Superintendent of the Coast Survey estimated that, at the rate of progress then being made, he would complete the original surveys of the Atlantic and Gulf coasts in about 12 years. The estimate was doubtless correct if all things had remained equal; but our Civil War was soon an interruption, and subsequently the necessity for re-surveys and the demand for more rapid extension in more important localities necessitated deferring a large part of the work on the Atlantic and Gulf coasts, that the forces available might be utilized in these other more important localities, and, consequently, the original surveys can hardly be said to be completed to-day. Another instance of the unreliability of such estimates can now be found in the Geological Survey. In 1885 Major Powell, the Director, stated that at the rate of progress of his work at that time he estimated that he would be able to complete the geological map of the United States, exclusive of Alaska, in a period not longer than 24 years;* but now Mr. Gannett estimates that the topographic work alone will require twice that period of time, and from one of Mr. Baker's tables I find that the average cost per square mile for this topography has been increased from $4.27 for 11 years to $6 for the last three years, and to $9.58 for the year 1893. If the estimates of the Geological Survey are to increase at this alarming ratio, we may well fear that the year 2373 A. D. will overtake even that progressive organization before it has completed the topographic survey, and that $120 000 000 will be a mere bagatelle in comparison with the actual cost.

Mr. Baker has restricted his estimates of the cost of the field work of the Coast Survey to the seven years since 1885. For reasons which

* Public Documents—Senate, 49th Congress, 1st Session, Miscellaneous Documents No. 82, pages 20 and 42.

I have before outlined it is impracticable to ascertain the exact cost of any branch of the work, even for this brief period; but an approximation could doubtless be arrived at by a thorough inspection of the accounts; this would prove or disprove Mr. Baker's statements. But for the present purpose it will be quite sufficient to show that he has fallen into a very serious error, which will, at least, prove the negative. An endeavor to divide the work into three classes for the past few years, with sufficiently complete data before him to show the nature of the expenditures, is a serious mistake that seems to be hardly justifiable in preparing an estimate that was to be compared with the cost of other surveys which include no expenditures beyond the manuscript map, as shown in his final table of the cost of different surveys in the United States; but he has committed a much more marked oversight in charging all the expenses of the subsidiary triangulation to topography, for his experience as an officer of the Coast Survey for a number of years should certainly have taught him that triangulation is as essential in the execution of hydrographic work as it is in topographic work. The percentages, therefore, that he presents are entirely untrustworthy. If we should transfer to hydrography one-half of the expense of triangulation that he has allotted to topography, it would reduce his final estimate of the cost of topographic work about 24 per cent. What further reduction would be made by allotting expenditures to the various classes to which they should properly belong I am unable to say, and must leave Mr. Baker or some other kind friend the task of ascertaining.

The detailed survey of the District of Columbia was conducted jointly by the city government and the Coast Survey, and I am not aware that the total expenditure has ever been compiled, although there should be no difficulty in ascertaining the fact if required. I have already described the work, but may as well lay stress here on the fact that the elevations are determined with the same precision as the linear distances; and in view of the cost of all detailed work of this character of which we have any record, in which both the horizontal and vertical distances have been determined with absolute precision, I do not believe that even the estimate presented by Mr. Baker, for $5 per acre, would be excessive. In Appendix 16 of the Coast Survey Report before referred to are two letters from Mr. J. C. Olmstead, in which he gives the cost of a number of special surveys for park im-

provements and similar purposes, and in which he concludes "that the fair, ordinary cost for a quality of survey such as suits our purposes is generally about $5 per acre." But he states that such surveys range from $2.50 to $20 or more per acre, the larger price, of course, including much greater detail, such as shrubbery, boulders, etc., that may appear upon the surface. Statements of cost, to be intelligible, should embrace statements of the accuracy of the work with them, for the greater cost is usually entailed by greater accuracy. The appearance of the map is not always a safe guide to a judgment of its value, and I deprecate all comparisons that do not explicitly set forth the degree of accuracy the work represents.

Let the engineers in the country determine the character of the work that will best subserve general purposes, and we may be assured that legislators and those interested in the general welfare will appreciate their efforts and assist in securing its accomplishment.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

677.

(Vol. XXX.—December, 1893.)

THE USE OF LONG STEEL TAPES.—DISCUSSION ON PAPER No. 636.*

By ROBERT MOORE, CHARLES B. BRUSH, J. A. OCKERSON, O. B. WHEELER and H. M. MARSHALL.

ROBERT MOORE, M. Am. Soc. C. E.—I can give an incident from my own experience in regard to the use of steel tapes which I now use for all classes of work in preference to anything else. Some years ago, when it became necessary to construct the elevated line of the Merchants' Bridge Terminal, the railroad company undertook to place the foundations on which the superstructure was afterwards to be placed, and it became necessary to so locate them that the superstructure would go on without any misfits? I sent to the Phœnix Bridge Company, who were making the superstructure, and asked them to send me a steel tape which was accurately compared with their standards. In reply they told me that it was not their practice to make accurate measurements with steel tapes at all; but that they used wooden rods. So in place of the steel tapes they sent me some wooden rods which were accurately compared with the standard in use in their

* "Recent Experience on the U. S. Coast and Geodetic Survey in the Use of Long Steel Tapes for Measuring Base Lines." By R. S. Woodward, C. E. *Transactions*, Vol. XXX, page 81.

shop. I instructed the assistants to use these rods on the work. They reported, however, that, owing to the difficulty in making and maintaining perfect contact between the rods, their work did not check up as well as with the steel tape, and requested my permission to lay the rods aside. This was accordingly done, and the steel tape only used in placing the several thousand anchor bolts which were necessary in this structure. The results were entirely satisfactory, as in placing all the spans there was not a single misfit due to any failure in placing the anchor bolts. I am satisfied there is nothing superior to the steel tape for all purposes of engineering work.

CHAS. B. BRUSH, M. Am. Soc. C. E.—The foundations of all the structures for the North Hudson County Railway Company in New Jersey were located with steel tapes. There was no difficulty. The results obtained were satisfactory.

J. A. OCKERSON, M. Am. Soc. C. E.—In August, 1880, a base line for secondary triangulation was measured by the writer with a 300-ft. steel tape. This is thought to be the first secondary base measured with a long steel tape in this country. It was measured as an initial base for the secondary triangulation of the Mississippi River.

This base line is situated in Missouri, opposite the mouth of the Illinois River. It was 6 663 ft. long and level from end to end.

A constant and uniform tension in the tape was obtained by a fixed weight hung over a pulley.

Only one thermometer was available, hence only one was read. The tape was supported on the tops of stakes 1 in. wide at intervals of 25 ft. The stakes on which the marks were made were entirely detached from the straining stakes.

Two measurements were made under a bright sun with an average temperature of 94° Fahr. 2 ft. from the ground, and 123° with the thermometer lying on the ground.

Each measurement occupied nearly four hours, and the discrepancy in the two measurements was 1 in 73 000.

In 1891, a base was measured at New Boston, Ill. Length of base, 18 066 ft. Average time required for each tape length, 2.2 minutes. Discrepancy between first and second measurements, 1 in 759 000. The measurement was made before sunrise.

In 1892, a base line was measured at Rapids City, Ill. Length of line, 5 624 ft. Time required for first measurement, 41 minutes; second,

28 minutes. Discrepancy between first and second measurements, 1 in 594 917.

In 1893, three secondary base lines have been measured, viz.: At East Dubuque, Ill., length, 7 105 ft. Time required for first measurement, 68 minutes; second, 53 minutes. Discrepancy between first and second measurement, 1 in 346 965.

Cassville, Wis., length, 7 001 ft. Length of time of first measurement, 47 minutes; second, 37 minutes. Discrepancy between first and second measurements, 1 in 266 412.

Prairie du Chien, Wis., 5 312 ft. long. First measurement made in 45 minutes, second in 35 minutes. Discrepancy between first and second measurements, 1 in 265 000.

None of these measurements were made in the sun.

The tape used in the above measurements, except the first, is 300.03851 ft. in length with a tension or pull of 16 lbs. at a temperature of 62° Fahr., and supported at intervals of 30 ft.

The dimensions are 0.0246 by 0.1225 in.; area of cross-section, 0.003014 sq. in.

The tape was made by Buff & Berger.

The preparation of a base line one mile long requires about 500 ft. of lumber for supporting and straining stakes. It requires the labor of six men for about two days, to clear the line, set the stakes, and establish the grades. The measuring is best done with one man at each 30-ft. length, one marker and one helper at the rear end, one marker and two helpers at the front end, and one man at each of three thermometers, or a total of 16; a much smaller number could do the work very well.

In the survey of the Mississippi River, the results obtained with the long steel tape have been highly satisfactory, and the small cost of a line measured in this way makes it practicable to use more lines and thus materially reduce the magnitude of the errors in a chain of triangulation by having more frequent checks. It has therefore been decided to have base lines at intervals of about 25 miles or about 12 triangles apart.

In the enumeration of base lines given above, the figures are only given in round numbers, as the final reductions of some of them have not yet been made.

With proper appliances and careful manipulation a high degree of accuracy can be attained with the steel tape, and it should be adopted

without hesitation for all base lines, except a very few standard primary bases to be used as initial and final check lines.

The important thing is the correct temperature of the tape. The latter is very sensitive and responds to changes of temperature more quickly than the ordinary thermometer.

O. B. Wheeler, M. Am. Soc. C. E.—Our use of the long steel tape under the Missouri River Commission has been to transfer, as it were, a carefully measured base line to other parts of the country. In other words, we standardize our steel tape by measuring a primary base line by the same method and apparatus that we use in measuring a new base line. By measuring at night we get a good control of the temperature disturbance. I have long believed that a base line can thus be transferred with all sufficient accuracy for any system of primary triangulation, since the extreme accuracy of a measured primary base line is soon lost through the angle measurements.

It is proper, perhaps, to discard the friction hooks, especially if the standardizing is done without their use, but I think the automatic "stretcher" or "adjuster" should not be discarded. Autometry is more in the line of progressive science. The "adjuster" as used by us, and described and illustrated in the report of the Missouri River Commission for the year 1886, consists, in principal, of equal lever arms joined at an angle of 90° in a knife-edge bearing at the juncture. The horizontal arm carries the weight and a spirit-level, while to the vertical arm the tape is attached. The bearing for the knife-edge is on a platform upon posts, and is controlled by slow-motion screws.

The exact pull to a small fraction of an ounce is found by placing the weight upon a platform scale, while its support is fixed in its proper place on its arm, the knife-edge being in its bearing and the level showing that the horizontal arm is horizontal. The rear end of the tape is fixed to a stretching post, and controlled also by a slow-motion screw.

In measurement, in moving forward, the "adjuster" is rapidly dissected, and as great rapidity is obtained as that shown in the paper under discussion. It is possible also to eliminate the "lag of thermometers," when measuring during a continuous fall (or rise) of temperature, by measuring in the reverse direction, the rear end of the tape becoming the forward end, to obviate the necessity for more platform stakes.

To prevent as much as possible the disturbance by wind, dew and dust, I would recommend a much lighter or narrower tape. The piano wire, as used by Professor S. W. Robinson in 1873, is the ideal, only that the flat wire is preferable to the round, that any twist may be detected.

I am happy to have it demonstrated that the long steel tape is applicable for the measurement of the primary base line, and I would suggest that if ever the arcs of meridian in distant countries are to be remeasured, that the base lines be measured with this easily transported apparatus.

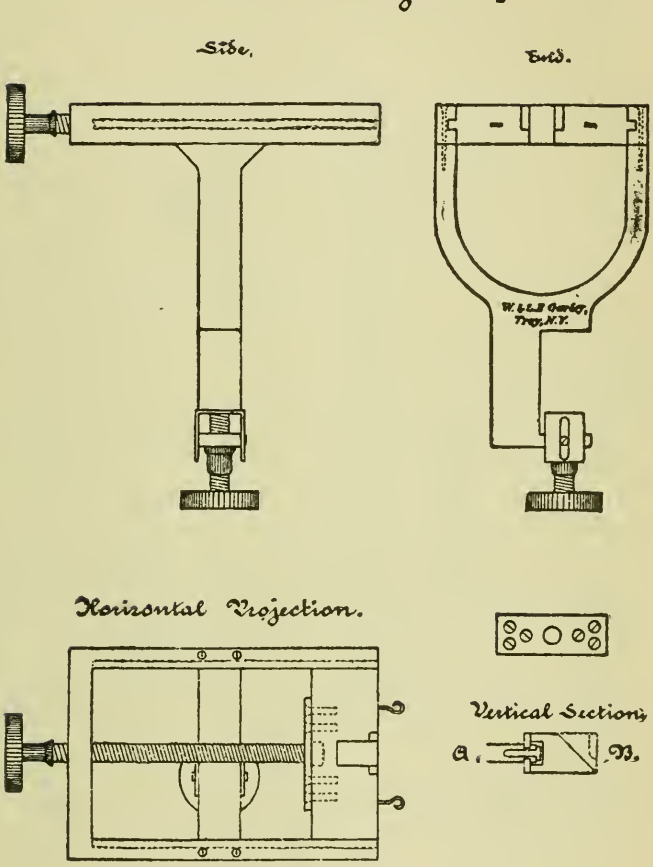
H. M. Marshall, M. Am. Soc. C. E.—Professor Woodward has added very strong evidence as to the efficiency of long steel tapes in base measuring, and, as imitation is the sincerest flattery, he has testified his highest appreciation of the good work done on the Mississippi and Missouri Rivers down to the minutest detail. The ice-packed bar proposed by Mr. E. S. Wheeler, the zinc plate of Mr. O. B. Wheeler, and the spring balance of Professor J. B. Johnson, all come in for commendation. It is, however, a little surprising that he failed to use the wire hooks on the nails to support the tape, for they would have saved using the tape after the fashion of a score line to adjust the friction.

In the year 1889, when triangulation was begun on the Red River Survey, it was not considered necessary in the U. S. Engineer Office at Vicksburg to make any particular investigation as to the accuracy to be attained in measuring bases with steel tapes. This was partly because a small tertiary system had to serve for practical purposes of the work, a secondary triangulation being contemplated only in the dim future; and partly because, being a branch of the public service somewhat connected with the river commissions, their experience was quite acceptable at its face value.

With the value of the method proven, only improvement in the details of the appliances was to be desired. Mr. O. B. Wheeler's apparatus used on the Missouri River to apply tension was as much preferable to that on the Mississippi River as a steel yard is to a spring balance in weighing, and only needed to be made of metal to be perfect. This was done, as shown on pages 643 and 644, and described as follows:

(1) Four tension frames.

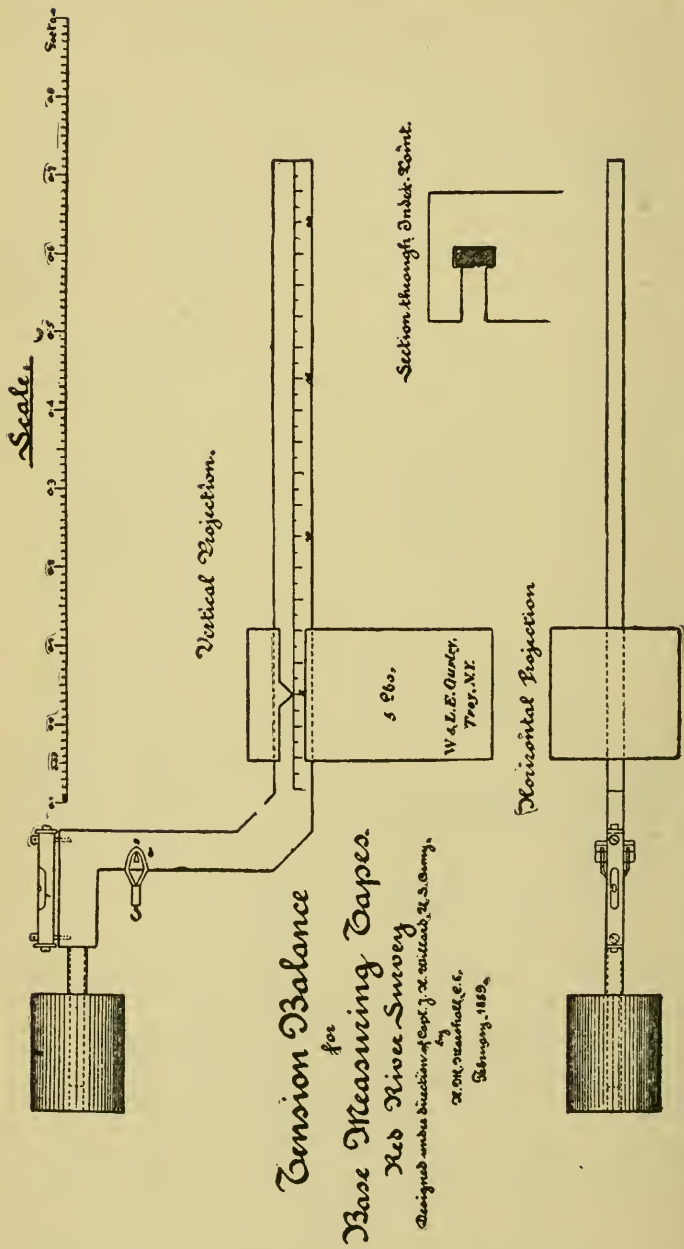
(2) Two balance beams.



Tension Balance
for
Base Measuring Tapes,
Red River Survey.

Designed under direction of Capt. J. N. Willard, U. S. Army,
by
N. M. Marshall, C. E.
February, 1889.

Scale.


Scale.
Vertical Projection.
Section through Index Point.
5 Lbs.
W. & L. E. Gurley,
Troy, N.Y.
Horizontal Projection
Tension Balance
for
Base Measuring Tapes.
Red River Survey
Designed under direction of Capt. J. H. Willard, U. S. Army.
by
H. M. Marshall, C. E.
February, 1889.

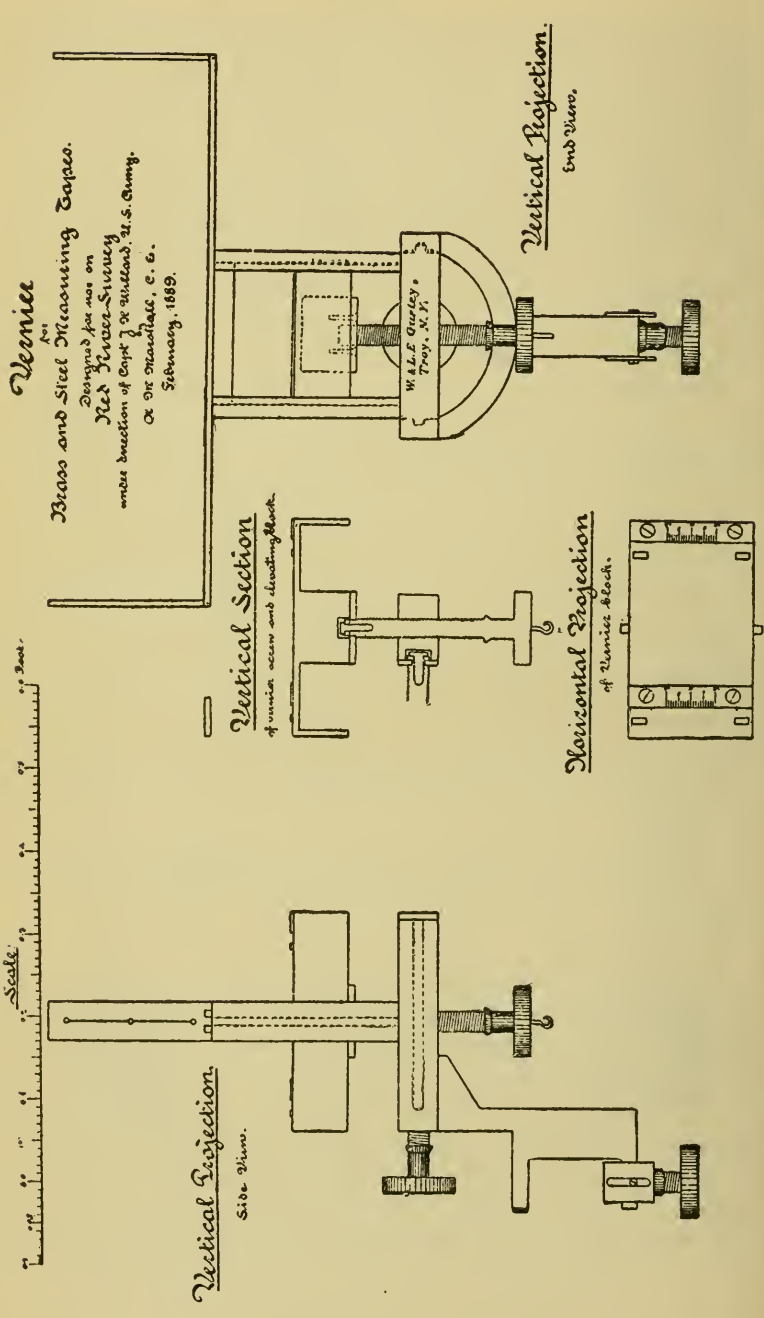
The tension frame consists of a clamp and frame of brass shown in vertical projection side and end and horizontal projection, page 643 The clamp has two arms above it, to which is screwed the three-sided rectangular frame. A screw working in one end moves the brass block, which is guided by flanges sliding in grooves in the sides of the frame. The screw is attached to the block, as shown in section A B, by a collar, so that it will not be disengaged by turning. Two small hooks are screwed in the front of the block, to attach the tape-chains when the frame is used at the rear end of the tapes. A triangular space is cut out of the block, and in the sides adjacent are cut seats for the knife edges r of the balance beam to rest in.

The balance beam, shown in side view, and projection, page 644, is brass, and has steel knife-edges r on which it rests, and knife-edges s, about which turn the stirrups of the steel hook to which is attached the front end of the tape-chain. On top of the upper short arm of the beam is a small level vial in a case attached by adjusting screws to the beam. On the end of this short arm is a screw on which works a counterpoise, to balance the beam without the pea. The long arm of the beam is divided, so that the mark 5 is the same distance from the knife-edges r, on which the beam rests, as are the knife-edges s, about which the stirrups turn, and indicates a tension equal to the weight of the pea, and twice that distance a tension twice the weight of the pea. The pea weighs 5 lbs., and its mass is so distributed that the center of gravity lies in a vertical plane perpendicular to the face through the index point.

The method of marking tape ends was not considered satisfactory, and in lieu of pieces of zinc two verniers were substituted, as shown on page 646.

(3) Two verniers.

The vernier, shown in vertical projection, side and end views, also vertical section and horizontal projection of the vernier block (page 646), consists of a clamp and frame similar to the frame of the tension-balance, with the addition of a hollow cast-block. This block is attached to a slow-motion screw for raising it under the tapes after they are stretched, and is guided by flanges sliding in grooves in two vertical arms fastened to the other brass block, which is capable of transverse motion. The hollow cast-block has on its upper face two metal graduated strips laid flat, with stops near the ends, to hold the tapes close


Vernier
for
Brass and Steel Measuring Tapes.
Designed for use on
Red River Survey
under direction of Capt. J. N. Macomb, U.S. Army.
& Dr. Marshall, C. E.
February, 1889.
W. & L. E. Gurley,
Troy, N. Y.
Vertical Projection.
End View.
Vertical Section
of vernier screw and sliding block.
Horizontal Projection
of Vernier block.
Vertical Projection.
Side View.
Scale.
Feet.

General View

Rear End

Front End

to the vernier. Verniers are marked 0 at the middle and are set exactly opposite. The graduation extends 9 mm. each way from the zeros, and is divided into ten parts, so that the reading is to 0.1 mm. In the grooves of the vertical arms slips a flange fastened to a bar carrying folding compass-sights, which enables the vernier zeros to be brought at right angles to the base line. In the center of the head of the vertical slow-motion screw is a small hook for attaching a plumb bob.

These verniers do not admit any referring in the future to the observations except through the record, and neither does a transit, to the angle measured by it; but, like the transit, the verniers permit independent measurements to be repeated at will. The apparatus was made by W. & L. E. Gurley, and is contained in a box 10 x 14 x 20 ins., the whole weighing about 40 lbs. The tapes—for it was designed to use a steel and brass tape—were made to order, a little more than 100 m. long, 3½ mm. wide, and one-tenth as thick. The breaking load of the brass tape is 96 lbs., and of the steel, 387 lbs., by tests made at the Watertown Arsenal. The graduation was done in the U. S. Coast Survey shops at Washington, in November and December, 1889, the steel tape being copper-plated at the ends to receive the engraving. This graduation extends both ways; from the standard marks near the tape ends over a space of 2 dcm. divided to millimeters. The numbering runs both ways from the zeros at the standard marks. The tapes are kept and transported on reels about 15 ins. in diameter, made like sheaves with ⅜-in. opening, turning in a frame by a crank and handle. This is not quite satisfactory, and they will be wound on drums.

Measurements were conducted in the usual way, excepting alterations brought about by the use of the verniers. The programme is as follows:

Beginning at the rear end of the tape, two stakes are driven about 3 ft. back of the initial point of the base. On top is nailed a piece of plank, which makes a table to clamp tension frames to, one on each side. Two stakes are next driven level with the first two stakes, but 2 ins. to the right of the line, about 9 ins. forward and back of the initial point of the base. On these is nailed a plank for clamping the vernier. Along the line, at intervals of one-tenth the tape length, stakes are set, with nails driven in the rear face of the stake, the heads being a little lower than the points; the nails are kept about level by the eye. At the front end of the tape, stakes are set for tables, as at the rear end; and in addition a table is set on line about 3 ft. forward of the end of the tape and 2½

ins. lower than the vernier table, for clamping the tension frames o. the balance beams. Elevations are taken on the vernier tables and heads of nails with a level. An observer, a recorder, and one assistant at each end of the tapes, and nine men at intervals of one-tenth the tape length, place the tapes one on each side of the stakes supported by the suspension hooks hung on the nails. The assistants adjust the clamps near the tape ends, and hook on the chains, clamp the tension frames to the straining tables, one on each side, and by means of the chains attach the rear ends of the tapes to the frames and the front ends to the hooks on the balance beams, rest the beams on the knife-edges, and slip the pea on the beam and set it to the required pull ; adjust the tension by the slow-motion screws of the frames till the beam is balanced and the level centered. The observers clamp a vernier to the table at each end of the tape. At the rear it is set by the plumb over the initial point, aligning by the compass sights, raised under the tapes by the elevating screws, and adjusted by the transverse screws till the tapes lie easily alongside the verniers. With pocket rule the rear observer measures the height of the vernier above the table. The observer at the front end cries, "Ready! Read steel !" Both read the distance in millimeters on the tape from its zero to the vernier zero and tenths of a millimeter on the vernier, and have the recorders make the entry. The observer at the rear end replies, "Read !" The observer at the front end cries, "Read brass !" Both read as before, and the observer at the rear end replies, "Read !" The front observer cries, "Shift tapes forward !" and assistants by means of the slow-motion screws move the tapes bodily several centimeters along their length, the men at intermediate stakes being careful to keep the supporting-hooks swinging free by tapping them lightly. The front observer again cries, "Ready !" and the operation is repeated. The tapes are then shifted back beyond the initial reading, and readings taken again.

In the reduction the mean of all the readings for each tape is taken to obtain the distance between vernier zeros.

For the next tape length the front observer calls out "Make ready!" the assistant at the front end lifts the balance-beams and slips off the peas, both assistants unhook chains and unclamp tension-frames, men along the line lift tapes and suspension hooks, the rear observer unclamps the vernier and sends it forward by his recorder to the front recorder, the front observer leaves his vernier clamped and calls "Forward, march !" The remaining part of the base is measured in the

same way, except that after moving forward the rear observer does not change the position of his vernier, unless he may have to alter the elevation by the elevating screw. In reading and recording, forward is counted from the tape zero in the direction the base is measured. At the front end of the tape, forward readings are plus, and backward readings are minus. At the rear end, forward readings are minus, and backward readings are plus. The check on readings in the field is that the first reading at front end minus the second reading at the front end must be nearly equal (within 0.5 mm.), the first reading at the rear end minus the second reading at the rear end, and must be of opposite sign; in other words, after the tapes are shifted, the shift must show the same amount and of opposite sign at the two ends of the tape; otherwise the readings are wrong.

To ascertain the length of the tapes, the Coast Survey base at Delta, La., was measured with the steel tape alone, on three nights in April, 1889. This base had been measured by means of 6-m. contact slide rods by the U. S. Coast and Geodetic Survey in 1878, and the length between monuments was furnished Captain J. H. Willard in manuscript by especial request for our purpose. The following table shows the field record on the first night for the first half dozen tape lengths:

DELTA BASE.

Observations for Standardizing Steel Tape—April 9th, 1889.

FRONT END.

TAPE No.	READING. Forward. +	READING. Backward. −	Thermometer No. 2.	Time.	Mean reading.	REMARKS.
	Millimeter.	Millimeter.	Degrees.	P. M.	Millimeter.	
1...	67.1	.....	66.1	8.22	+ 27.7	T. C. Thomas, observer. W. H. Polk, recorder.
	4.6	.....	68.0			
	11.3	.....	68.4			
2...	39.1	.....	66.8	8.40	+ 41.2	Thermometer No. 2. At 70 m. from rear end is U. S. Signal Service compared thermometer No. 791.
	32.0	.....	66.9			
	52.5	.....	66.5			
3...	2.8	.....	63.1	8.57	+ 7.7	
	3.1	.....	63.1			
	17.1	.....	62.5			
4...	.....	1.3	60.2	9.23	+ 0.6	
	1.5	.....	58.2			
	1.5	.....	58.1			
5...	43.9	.....	60.0	9.50	+ 35.4	
	36.4	.....	59.6			
	25.9	.....	59.3			
6...	49.7	.....	57.8	10.04	+ 58.8	
	59.5	.....	57.7			
	67.2	.....	57.7			

REAR END.

TAPE No.	READING. Forward. +	READING. Backward. −	Thermometer No. 1.	Time.	Mean reading.	REMARKS.
	Millimeter.	Millimeter.	Degrees.	P. M.	Millimeter.	
1...	19 3	.....	68.0	.....	+ 19.4	T. G. Rombauer, observer. C. H. Schermerhorn, recorder.
	.....	42.0	67.8			
	.....	35.5	68.8			Vernier above table 71 mm.
2...	26.0	.....	68.5	.....	— 28.6	Thermometer No. 1. At 30 m. from rear end is U. S. Signal Service compared thermometer No. 1522. Vernier above table, 64. mm.
	20.0	.....	68.8			
	39.8	.....	68.8			
3...	.....	8.8	64.3	.....	+ 4.1	Vernier above table, 70 mm.
	.....	8.7	64.3			
	5.3	.....	64.9			
4...	14.1	.....	63.3	.....	— 14.1	Vernier above table, 57 mm.
	14.1	.....	62.1			
	14.2	.....	62.8			
5...	9.3	.....	61.0	.....	+ 0.8	Vernier above table, 70 mm.
	1.8	.....	60.9			
	.....	8.7	60.8			
6...	14.3	.....	59.0	.....	— 22.8	Vernier above table, 57 mm.
	23.5	.....	59.0			
	30.7	.....	58.9			

The tape length was evaluated from the three nights observations, by use of the formula—

$$L = \frac{B + \frac{\Sigma h^2}{2l} + \frac{\Sigma h^4}{8l^3} - \Sigma\,(s + s^1).}{n + e\,\Sigma t}$$

in which B is the length of the base, h and l the difference in height and distance between supports, s and s^1 the readings at the front and rear ends of the tape, n the number of tape lengths, e the coefficient of expansion of the tape, and t the temperature, Fahrenheit, minus 60°.

The relative lengths of the steel and brass tapes were ascertained by 51 simultaneous readings of the two tapes stretched side by side, at various times under 16 lbs. tension, and supported as in measuring a base, the temperature varying from 40.4 to 88.° Fahr.

The result of each observation being written in the general form—

$$t = \frac{L - L_1 - (b + b^1) + (s + s^1)}{e_1 L_1 - eL}, \text{ or}$$

$$t\,(L_1 e_1 - Le) = L - L_1 + [(s + s^1) - (b + b^1)]$$

putting $y = (s + s^1) - (b + b^1)$; $x = t$; $a = L_1 e_1 - Le$; $c = L - L_1$ the equation becomes $ax = c + y$.

When x and y are taken as the mean of all the observations

$$c = ax_o - y_o$$

and taking the difference between the mean and each observation

$$a\,(x - x_o) - (y - y_o) = 0.$$

By multiplying through by $(x - x_o)$, that is, giving each observation weight according to its range in temperature,

$$a\,(x - x_o)^2 - (x - x_o)\,(y - y_o) = 0.$$

Summing the results of all observations,

$$a\,\Sigma\,(x - x_o)^2 - \Sigma\,(x - x_o)\,(y - y_o) = 0$$

$$a = \frac{\Sigma\,(x - x_o)\,(y - y_o)}{\Sigma\,(x - x_o)^2}$$

L_1, e_1, b and b^1 being the length, coefficient of expansion, and readings of the brass tape. The length of the tape being known, the length of a base is given by the formula—

$$B = n\,L + e\,L\,\Sigma\,t - \frac{\Sigma\,h^2}{2l} - \frac{\Sigma\,h^4}{8l^3} + \Sigma\,(s + s^1)$$

and the temperature by the formula—

$$\Sigma\,t = \frac{n\,(L - L_1) - \Sigma\,(b + b^1) + \Sigma\,(s + s^1)}{e_1 L_1 - e\,L}$$

When $L = L_1$

$$\Sigma\,t = \frac{\Sigma\,(s + s^1) - \Sigma\,(b + b^1)}{(e_1 - e)\,L}$$

To check the length of the steel tape, as determined at the Delta base, measurement was made of the U. S. Coast and Geodetic Survey base at Lum Point, La., during nights in February, 1892. The length between monuments was furnished again in manuscript with that end in view. This base was not commensurable with the tape, and 3.75 m. were deducted by measuring back with a precise level rod, and the remaining portion measured with the steel tape. The length of the base as measured by the tape was 1 921.8223 m., and was given by the Coast Survey 1 921.823 m. To get a further independent check, the base was extended five tape-lengths, using the tape as short as possible; that is, with the scale readings all minus, and then base and extension were measured as one, using the tape as long as possible. Two measurements were made, and while the mean is quite close to the first determination, the range is wide.

The following results serve to show the discrepancies in the several measures:

DELTA BASE.

April 9th, 1889	27 L +	92.1 mm.
April 10th, 1889		91.7 "
April 11th, 1889		91.3 "

LUM POINT BASE.

February 1st, 1892	19 L +	10 175.3 mm.
February 4th, 1892		10 158.9 "
February 5th, 1892		10 187.6 "

The mean of the three determinations of the tape-length at Delta is 100.61300 m., and at Lum Point 100.61310 m.

To compare these measurements with those on St. Albans base, the discrepancies on Delta base should be increased $\frac{38}{27}$. The discrepancy between the first measurement of Lum Point base and either of the others must be increased $\frac{38}{24}$, and between the two last by $\frac{38}{29}$, the measurement having been extended beyond the base monument. This is on the principal that the error increases as the distance, and not as the square root of the distance, when the distance is small. That is, the law of probabilities only holds for a great number of possibilities, and the method of least squares is not properly applicable to a few observations.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

678.

(Vol. XXX.—December, 1893.)

TERRESTRIAL MAGNETISM IN NORTH AMERICA. —DISCUSSION ON PAPER No. 637.*

By HENRY GOLDMARK, R. S. WOODWARD and CHARLES A. SCHOTT.

HENRY GOLDMARK, M. Am. Soc. C. E.—Is the method devised by Gauss and Weber for measuring magnetic intensity still used?

Mr. R. S. WOODWARD.—It is still used, with some improvements.

A MEMBER.—Are there any local sources of disturbance in making measurements of magnetic force?

Mr. WOODWARD.—There are such sources. In general, however, it is arranged to avoid them. Some difficulty is nowadays found in establishing permanent observatories, because there is so much electricity generated for various purposes. A few years ago, it was found that an observatory at San Antonio, Tex., was affected by an electric plant some distance away. This plant produced currents along water pipes laid in the ground near the observatory, and the disturbing effect of those currents on the magnetic instruments was so great that the observatory had to be removed to a new site. Great pains are always taken to avoid such sources of disturbance.

* "Terrestrial Magnetism in North America." By Charles A. Schott, Assistant U. S. Coast and Geodetic Survey. *Transactions*, Vol. XXX, page 108.

A MEMBER.—In a case which has come to my notice where observations were made near a locomotive shop, the disturbance to the magnetic needle was very great.

A MEMBER.—How great a distance is it essential to remove an observatory from sources of disturbance?

Mr. WOODWARD.—No general answer can be given to this question. Each case must be considered by itself. In refined work, the magnetic observatory is always placed at a safe distance.

Mr. CHARLES A. SCHOTT.—Apart from the disturbing effect on magnetic observations of terrestrial magnetism by the trolley and underground electric wire roads, by the storage car lines, by the electric light and telephone wires and by iron or steel structures of all kinds, which influences can generally be avoided by selecting a proper place for making the observations, there are also physical sources affecting the regularity of distribution of the magnetism of the earth's crust.

For convenience sake we may distinguish two kinds of disturbances, *i. e.*, those which appear quite local, covering only a few square miles, and which may be due to a single protruding disturbing mass, generally of the same polarity as that of the hemisphere in which they appear, and secondly, the larger disturbance extending over hundreds of square miles, distributed generally in one or more ridges, and, of course, closely connected with the geological formation. The tracing out of isabnormal magnetic curves, defining and measuring the disturbed localities, still remains to be done.

The investigation of the secular changes of magnetism has become more difficult on account of the impracticability, in most places, of revisiting the old stations.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

679.

(Vol. XXX.—December, 1893.)

TREATMENT OF METALS FOR STRUCTURAL PURPOSES.—DISCUSSION ON PAPER No. 640.*

By L. L. BUCK, A. E. HUNT and J. E. GREINER.

L. L. BUCK, M. Am. Soc. C. E.—Naturally this paper is devoted to the treatment of steel. The treatment as well as abuse of iron as a structural material has been so thoroughly discussed that it would seem there is very little more to be said about it.

Steel has so largely superseded iron and has come into use so rapidly that our knowledge of its proper treatment has hardly kept pace with its adoption; hence any paper written by a man who has had as extended experience in working it as Mr. Christie must be a valuable contribution to engineering literature.

A reference to the paper in question shows such expectation to be fully justified; it is worthy of careful study by engineers and should be fully discussed. There would not be sufficient time for each member to discuss all the points of the paper. I have therefore decided to confine my discussion to such points only as my own experience has enabled me to form some opinion upon.

We are told that steel is a crystalline metal, and that good iron is fibrous.

* "The Treatment of Metals for Structural Purposes." By James Christie, M. Am. Soc. C. E. *Transactions*, Vol. XXX, page 155.

As regards steel, generally, this is doubtless true, and whatever error arises from treating it as a crystal will be on the side of safety. But iron, I take it, is fibrous only from the fact that when the puddle ball is passed through the rolls the particles of pure iron are kept more or less separated from each other by impurities, and these pure particles are elongated into fibers overlapping one another and welding together at numerous points. If for any reason a crack is started in such a material, it may be confined to but a fine fiber; while if a crack is started in a perfectly crystalline bar of steel, it would extend throughout the piece. Yet I have experimented with specimens of low carbon steel, by subjecting them to a slight nicking and bending test, and where the crack has started from such a nick, it has in many cases been arrested very soon and allowed of the specimen being closed together without further cracking. It was natural to suppose that in such a case the crack must have encountered a seam, beyond which was a surface of such extent as to allow of its stretching sufficiently to satisfy the bending. I have been told that this was an exploded theory, but exploded or not, the application of the microscope to such specimens showed such a surface, and I believe the crack will continue till it encounters such a seam. I have supposed that they must be caused by very fine bubbles which served to make the rolled material, in a measure, fibrous, as is done by the impurities in iron. If of appreciable size they would have a bad effect, where the bar was to be upset and forged, as in forming an eye-bar head or screw-end; but if very fine, they might arrest an incipient crack caused by shears or punch.

Mr. Christie states that low carbon steel is less liable to be cracked in punching or shearing than the higher sorts and that thin plates are less liable to it than thick ones. This is doubtless true. In short, anything which necessitates increased pressure in these operations, as, for instance, dull shears or punches, may cause cracks.

I am in favor of reaming all holes punched and all edges sheared in important steel members, and agree with Mr. Christie that such reaming should be done after the parts of a member have been assembled. Even with iron riveted work, I would have all holes reamed which did not agree closely enough to admit the rivet.

Most of us have seen the punched pieces of a member assembled when it did not appear possible to make the holes agree. But a gang

of men with drift pins and sledges would soon torture the parts into, apparently, happy family conformity. It would then go to the riveting machine and probably revenge itself upon the manufacturer by causing the machine and gang to be idle a quarter to a third of the time in drifting holes where the parts had been displaced, while other holes had been drifted. Had a gang of men been employed with hand reamers where the rivets would not enter, there would be no delay at the riveter and there could be no question as to the superiority of the work. Removing the sharp edges of the drilled or reamed holes by a slight countersink will not only benefit the rivet, but may prevent the starting of a crack in the plate as well.

As to driving field rivets, I hope the time will soon come when, by proper design of work and improvement of riveting machinery, most of such rivets can be machine-driven. When the stiffening trusses of the Niagara railway suspension bridge were erected, in 1880, I required all chord-splice rivets to be driven with 8-lb. to 10-lb. sledges, as the heavy sledges had an effect in upsetting the rivet very similar to that produced by pressure. The results have substantiated the theory. The Niagara chords were 80 000 lbs. steel, and have been subject to reversing stresses.

I only speak of this Niagara structure because it was some 15 years ago that the material was selected and it was a time when we did not know as much about steel as we do now, and, in fact, we knew, most of us, very little about it. It was something over 80 000 lbs. ultimate strength per square inch, and of course we could not get as good steel at that time as we can now. I had to reject 40% of all the plates that were tested. The steel was used in the chords of the bridge. They have been subjected to enormous stresses. In one case where a plate cracked and another was put in, I could not get a steel plate; and so had to put on an iron plate in place of it; the steel one was $\frac{1}{2}$ in. thick, the iron plate I made $\frac{1}{16}$ in. thicker. The first train that went over it pulled the plate in two. The other had been subjected to the very heaviest stresses for 14 years.

A. E. Hunt, M. Am. Soc. C. E.—Quoting from Mr. Christie's paper: "It is not intended here to discuss the quality of steel as determined by the component elements, a subject which has received ample attention at the hands of competent investigators. Suffice it to say that it is generally recognized that phosphorus and sulphur exercise an influence entirely evil, especially the former in its effects on material

when in service. If the manufacturer could banish them entirely, he would do so."

I would take exception to these statements as of broad, general and universal application, so far as my judgment goes. This is undoubtedly true regarding tool and other high grades of crucible steel, where there is a high percentage of carbon present, but I do not think it is true with reference to structural steel that has a carbon content ordinarily of less than 0.2 per cent.

Pure iron is a weak metal. I am not sure that it is as strong as the other metal in which I am interested—aluminum; that is, a tensile strength of not over 30 000 lbs. per square inch. It is only when a small proportion of other elements is added to it that the strength of wrought iron or steel is given to pure iron. When the percentage of carbon runs down very low and there is not some marked amount of phosphorus present, the steel produced is very low in tensile strength—below that of good wrought iron; that is, below 50 000 lbs. tensile strength per square inch, a material which is so soft that the unit strains ordinarily applied upon structural steel cannot be safely applied to it.

To counteract this weakness of very low phosphorus, manufacturers of basic open-hearth steel who have made their metal with less than 0.01% phosphorus have been obliged to add a considerably larger proportion of carbon, in order to obtain the tensile strength ordinarily required in structural material.

To get a steel of 60 000 lbs. tensile strength with the phosphorus below 0.015%, manufacturers of basic open-hearth steel in this country have found it necessary to run the carbon up to 0.3 per cent. Such steel is much more liable to take a temper by being cold-rolled or suddenly plunged into water when heated red hot. It is, according to my experience, a much more "tender" metal, in the way of being susceptible to changes, due to varying conditions to which it may be subjected, than steel which is lower in carbon and higher in phosphorus.

In the basic process in their efforts to reduce the phosphorus to as low a content as possible, carbon has often been run very low indeed in the melting furnace, and the steel has become liable to take up occluded gases and to be weak and rotten. I do not think, under ordinary conditions, that the general practice which has been widely

quoted, that phosphorus exercises an influence "entirely evil" in its presence in steel, and the manufacturer would banish it entirely if possible, is correct, or that the carrying out of this idea is good practice.

Undoubtedly the native ores that have produced stock for the manufacture of steel that was low in phosphorus have been of the best quality. I especially refer here to the materials producing the excellent results in Norway and Swedish irons. In a general way, also, the lower in phosphorus that structural steel can be obtained by the acid process, the better, for we have not yet found in this country ways of producing such metal with less than 0.02 or 0.03% phosphorus, except in very rare instances.

While I would advise boiler plate and steel of similar specifications to call for a maximum of, say, 0.03 or 0.04% content of phosphorus, I think that when we have gone to this limit for structural steel the specification will be as rigid in this matter as is wise; and, on the manufacturer's side, when he has produced a steel with phosphorus as low as 0.03%, he has gone as far as he need in purifying his metal in his endeavors to get a good structural material.

There are other causes for the production of poor steel beyond the content of phosphorus which can be better looked after than endeavoring to run the phosphorus lower than 0.03 or 0.04 per cent.

J. E. Greiner, M. Am. Soc. C. E.—The author states that public confidence is now fairly established in the trustworthiness of structural steel; that this confidence has been brought about by experience. He could have added that, while the reliability of steel of certain grades and processes has gained the confidence of the engineers who are called upon to use it, and the mechanics who manipulate and fabricate it into structures, the old-time respect for wrought iron has just as surely lost its hold and is gradually being crowded out.

It must be admitted that when steel was first introduced in the construction of bridges, our knowledge was limited and the metal was continually under suspicion, inspectors and chemists subjected it to every conceivable kind of treatment to satisfy themselves that when fabricated into a bridge of some 300 or more feet clear span, it would not fly into fragments like so much glass, upon the passage of the first locomotive.

The general idea of steel was a hard elastic metal, such as is used in

cutlery tools and wagon springs, liable to snap without warning especially in cold or frosty weather. One of the most prominent railroads in this country will not use steel in its bridges at the present time, in spite of the experience in its use and knowledge of its quality possessed by most bridge engineers. They still adhere to the wrought iron of the past and claim that steel is brittle, being liable to fly into fragments at the first shock, that it is liable to abuse in the shop, that it cannot be welded, that there is always a fear of small cracks having been started and not discovered, which sooner or later will cause a disaster, and, finally, that our knowledge will not justify its use—the same old objections which were advanced at its first introduction. This same company still uses iron eye-bars, relics of the past which are now made by but a few of the many firms engaged in bridge building, and which, although having from 50 to 75% excess of metal in the head, frequently break, not in the body of the bar as steel bars break, but through the head.

There was a time when wrought iron could be obtained, free from scrap, having an ultimate strength of from 50 000 to 52 000 lbs. with a correspondingly good elastic limit and elongation and a beautiful fiber. I have personally tested and inspected tons of this material and seen it fabricated in bridge structures. I have seen channel bars of the material made into posts with pinholes drilled near the ends, split open from the edge of the hole to end of web upon being roughly handled in the shop. I have seen I beam stringers made of this material after being in service a few years, suddenly split in their webs from the ends to a distance of 12 to 24 ins., flange angles of girders split along the roots by being struck a blow, any number of lattice bars and tie plates split by being punched too near the edges. All this was at a time when the mills agreed to live up to specification requirements and before the bridge builders got together and drew up specifications for qua ty of wrought iron, the result of which specifications has been to reduce the grade of material. Some railroad engineers being satisfied with the poorer grade, the mills will not at the present time undertake to make wrought iron such as specified by the Baltimore and Ohio specifications of 1885 except at a premium. Iron obtained now with an ultimate of 52 000 lbs. has very probably been doctored with steel scrap.

If, then, iron double rolled directly from the muck bars will so

easily split and crack, the poorer grade, such as now manufactured, has little to commend itself to the engineer, and why it is that some engineers will continue to give it preference over mild open-hearth steel can be attributed to a conservatism not well founded, as is clearly demonstrated by the experiment of Mr. Christie and many others.

The men who actually handle, punch, drill, shear and rivet up bridge members are the ones who favor a soft or medium steel to wrought iron. They have found that it can be bent cold into almost any shape without signs of fracture; that it can be used for lattice bars and tie plates where wrought iron would split out; that the former belief that it would fly into a thousand pieces if struck a severe blow was without foundation in fact; that when a girder dropped, the flange angles did not split along the rivets, as is the case so often with wrought iron; that eye-bar heads can be made to develop the full strength of the bars; that their work looked neater and cleaner when of steel. Those who know the least are the loudest in its condemnation.

Of course the foregoing remarks apply to steel made by the open-hearth process, either acid or basic, it being beyond question the best for structural purposes, because, as the author states, "the character of the product can be so readily controlled and predetermined."

Uniformity in quality is what is required, and the engineer naturally favors the process which will bring about these results, and although the managers of Bessemer plants will not readily acknowledge that their metal is not uniform from an engineer's standpoint, facts are arrayed against them which will require a long time and repeated demonstrations on their part to overcome.

The mills strive to produce a cheap product; they frequently produce mere jokes, having sacrificed quality and everything else to cheapness. Numerous instances are seen in the extremely poor quality of the Bessemer rails of heavy section now being made; rails which although weighing from 17 to 20 lbs. per yard more than those used five or six years ago, wear out and break, while the older rails are still doing good service. Several instances are on record where both Bessemer and open-hearth steels were used in the same girder which, upon being dropped on its flange, cracked the Bessemer angles clear across, while the open-hearth plates, which received the full impulse, merely

bent and were not otherwise injured. Another instance which recently came under my notice was the cracking of several basic Bessemer plates after having been punched and carelessly thrown to the ground. I have never seen nor heard of open-hearth steel acting in this manner, and am specifying it to the exclusion of other processes and wrought iron, in all railroad bridges now being built for the Baltimore and Ohio Railroad. Mr. G. B. Hazlehurst, M. Am. Soc. C. E., engineer in charge of bridge construction for the Baltimore and Ohio Southwestern Railroad, has also recently adopted the open-hearth steel in all bridgework. The specifications are as follows :

QUALITY OF STEEL.

General Character.—All steel must be uniform in quality and made by the open-hearth process. The finished bars shall be free from injurious seams, flaws or cracks and have a workman-like finish.

Phosphorus Limit.—The phosphorus in all melts must not exceed 0.08%, and certified analyses of all melts shall be furnished free of cost.

Melt Number.—All finished material must be plainly and distinctly marked with correct melt numbers.

Test Pieces.—Test pieces shall have a uniform sectional area of at least ½ sq. in. for a distance of 10 ins. Elongation shall be measured in a space of 8 ins.

Soft Steel.—When tested in specimens as above, soft steel shall give the following results:

Ultimate strength per square inch...	52 000 to 60 000 lbs.
Elastic limit.....	32 000 lbs.
Elongation in 8 ins.....	25%

Specimens must bend cold to 180° and close down upon themselves without sign of fracture. In tension tests, the fracture must be entirely silky. Holes punched 1½ ins. from edge of plate shall be capable of being enlarged 100% by drifting, without any sign of fracture.

Soft steel may be used under the same conditions as wrought iron, except that it must not be welded and that all bends be made cold or else the whole piece be annealed. The permissible strain per square inch may be taken 10% in excess of that given for wrought iron.

Medium Steel.—When tested in specimens, medium steel shall give the following results :

Ultimate strength per square inch...	60 000 to 68 000 lbs.
Elastic limit.....	35 000 lbs.
Elongation in 8 ins.....	20%

Specimens must bend cold to 180° around a curve whose diameter is equal to the thickness of piece tested, without sign of fracture. In tension tests, the fracture must be at least 80% silky. Holes punched $1\frac{1}{2}$ ins. from edge of plate, shall be capable of being enlarged $66\frac{2}{3}\%$ without sign of fracture.

Medium steel shall be used for pins and for eye-bars whose heads are to be formed by upsetting. It shall not be welded. All holes must be drilled. Bars must be carefully and properly annealed, permissible strain in such cases to be 12 000 lbs. per square inch.

Tests of Finished Bars.—Not less than 3% of the number of finished bars will be tested to destruction, and under this test they must develop the following :

Ultimate not less than ..... 58 000 lbs.
Elastic limit..... 30 000 "
Elongation, $12\frac{1}{2}\%$ of total length, center to center.

Steel Rivets.—In all cases where steel members are connected by rivets, the rivets must be of the best quality of mild steel.

It will be noticed in the above specifications that medium steel is specified for eye-bars and pins only, and that bars are to be annealed; and that soft steel is used for all other members and worked under the same condition as wrought iron, excepting that no welding will be allowed. No restrictions are placed upon the thickness of the metal which is to be punched, for the following reasons :

First.—The designs are made under personal supervision, and an effort is made to limit the thickness to $\frac{5}{8}$ or $\frac{3}{4}$ in. at the most.

Second.—Since it has been the accepted good practice to punch iron of this thickness and since mild steel is injured to no greater extent than iron and probably to a less extent, there is no more reason for limiting thickness in soft steel than in iron.

Some engineers specify that steel from $\frac{1}{2}$ to $\frac{9}{16}$ in. must be reamed or drilled, and, when less than this, it can be punched. It is difficult to keep all members within the above limits, and this means that most of the holes are to be reamed, since it is not practicable to separate the material according to thickness, one lot to be punched with a certain die, the other with a different die, and then reamed to the same diameter as the holes in the first lot. When reaming is done, it is done after the pieces are assembled, and holes through any thickness of metal will be reamed. While reaming will be of great benefit, it entails an additional expense, and the cost will eat up the saving in metal due to

the increased permissible strains, as compared with iron, as usually adopted; moreover, reaming in soft steel is more expensive than in either iron or medium steel, on account of the metal clogging the reamer.

The above specifications were written to meet the immediate demands, and may be modified before being inserted in the printed specifications of the Baltimore and Ohio Railroad, now under course of preparation. For instance, I believe in limiting phosphorus to .05% in basic and .08% in acid open hearth, believing that this will place both processes on an equal footing, and will give very good results. Phosphorus could be eliminated entirely, if the manufacturer so desired, but, of course, at an increase in cost.

Outside of the above, I do not think it necessary to make chemical requirements in specifications, believing that, when the physical tests are satisfactory, the chemical composition should rest with the manufacturer.

As stated by the author, the best results combined with the least expenditure can be obtained by punching all holes when vital strains are not transferred by the rivets, and by reaming all holes in important joints and connections, such as stringers to floor beams, floor beams to posts, connection of web members of riveted lattice girders with the chords and tension splices. This practice is followed in some of the best shops, although with the idea of making well-fitting field connections rather than with the intention of removing injured metal.

The numerous tests made by Mr. Christie are valuable, and present facts in a form very acceptable to the engineering profession. Many such tests have been made by inspectors and others, and all convey the same general characteristics. Structures as built during the last 8 or 10 years have generally been well designed and without failures from an engineering standpoint, and holes have been mostly punched, reaming being used merely to obtain better matching. Experience has demonstrated that working strains as adopted in this country for this class of work are amply low; in fact, structures built 10 years ago are now, in many cases, strained from 20 to 30% in excess of the specified permissible strains and are just as perfect under these increased loads as they were when first built. I should not consider a well-designed Pratt truss or plate girder as a cause for uneasiness even when

the calculated strains would run up to 50% in excess of the present practice.

If, then, our structures be made of a much superior metal and this metal be carefully manipulated, as suggested by the author, a very important question arises, although not strictly pertinent to the subject under discussion, viz., "how high can we safely take our strains?" I believe the weights of engines and trains have about reached their limit, the heaviest engines not doing the work expected. If this is the case, we could take a load, say, of 120-ton to 125-ton engine, followed by 4 000 lbs. as a standard, and run our strains up to 30% in excess of present practice and have a structure which would do its work. I would not be the first to take such a step, and others no doubt feel the same way, and it is probable that we will continue to waste tons of material until in the future we find our bridges built for 80-ton engines still doing the same service as those built for 125-ton engines, even after most of us had predicted their failure by being shaken to pieces.

The life of railroad bridges has been heretofore about 20 years, not because the bridges wore out under the traffic for which they were designed, but because the traffic has, during that period, increased to such an extent that the old bridges, mostly of inferior designs, were no longer to be trusted with the excessive loads. If, then, as stated before, we could positively fix a limit to the weight of engines and cars, there would no longer be occasion to adhere to the so-called factor of safety of 5, one-fifth of which factor goes toward providing for a future increased loading.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

680.

(Vol. XXX.—December, 1893.)

A PROPOSED METHOD OF TESTING STRUCTURAL STEEL.—DISCUSSION ON PAPER No. 641.*

By GEORGE S. MORISON, L. L. BUCK, T. S. WHITE, JAMES CHRISTIE, A. C. CUNNINGHAM, JOHN PURSER GRIFFITH and ALFRED E. HUNT.

GEORGE S. MORISON, M. Am. Soc. C. E.—There is one point in Captain Hunt's paper which has interested me deeply, though it is perhaps one of the lesser points. I will say something about it to start the discussion.

On page 187 he says that it is "perfectly practicable to equip the punches used at bridge and boiler shops in actual practice with some device for measuring the force or work done in punching; and by the aid of graphical devices a record can be kept of the characteristics of every piece of metal punched that goes into any structure." By following this we should have a perfect method of investigating the material in riveted members. Then, if we drew a specification which required the use of this registering apparatus and provided that the punches should always register within certain limits, every plate could be inspected and accepted as it was being punched. This would, I

* "A Proposed Method of Testing Structural Steel." By Alfred E. Hunt, M. Am. Soc. C. E. *Transactions*, Vol. XXX, page 181.

think, be an ideal method of testing, perhaps a little too near an ideal; but we may safely say that if this could always be done, no other inspection would be required. If we have a member which is perfect at every punched hole, I do not think it makes very much difference, of course within reasonable limits, what the metal is between the holes; furthermore if it is perfect at the holes it cannot be very bad between.

There are two or three objections, however, to the use of this method of testing. The first is that the testing machine has yet to be devised; it can be devised, but it is a new machine. The second is that I think in the best class of steel work it is better not to punch at all. We should fit our pieces together by planed edges in immediate contact, without any of the clearances which are now allowed for irregular rolling. When the pieces are fitted together in this way the holes should be drilled in the solid metal through the several pieces, in the actual positions which they will occupy in the finished structure. This is the only way to get first-class work, and it is the only way to get work free from initial strains.

Subject to these provisions it seems to me that Captain Hunt's paper is worthy of the most careful consideration; the method can undoubtedly be applied to all riveted members, but how are you going to apply it to either eye-bars or pins? The answer seems the simple one, that you cannot apply it at all. I know of no method of testing eye-bars which will give perfectly satisfactory results except that of taking every precaution to obtain uniform steel and then testing a number of the full-sized bars. As regards pins, there is a method of testing which I believe to be as good and useful as Captain Hunt's method of testing riveted plates would be. As I said before, I have advocated for some time drilling a hole through the center of the pin. In some pins made in one of the eastern shops the manufacturer voluntarily and without request bored the pins in such a way that a continuous round core was taken from the center of each pin. A core taken from the center of a pin is pretty sure to be the worst metal in the whole pin. If you will take a core from every pin and test it, both by bending and tensile strain, you will have a perfect set of tests for pins.

L. L. Buck, M. Am. Soc. C. E.—It has long been my opinion that a good place for an inspector of structural material is in the shop where the material undergoes the various operations of being made up into

the members of the structure for which it is intended. Much information can be gained by examining the chips removed by drill, lathe and planer, as well as by noting the action of the punch.

There is doubtless a relation existing between the resistance of the material in the testing machine and that presented to punching and shearing. If I understand Mr. Hunt's paper, he proposes to establish such relation, not for the purpose of dispensing with the testing machine, but to enable the inspector to obtain a more through acquaintance with the quality of the material, or, as it were, to make his method auxiliary to the testing machine.

That this may be done with valuable results by the use of the punch there is little, if any, doubt; but it appears to me that the method by shearing would present so many conditions dependent upon the form of specimen and its relation to the form and size of the cutting edges of the shears that it would be almost impossible to establish a relation between the tensile strength and the shearing resistance by that means.

The fewer the conditions to be noted in any method of testing, providing they are carefully selected and authentic, the better. Even with the punch there would be a variation with every change in size of punch, thickness of plate, proportion of size of punch to that of die and the sharpness of punch and die. The last two could be eliminated by specifying that the punch should have a given clearance in the die and that both should be kept sharp.

The shearing test would be useful for establishing the relation between shearing and tensile resistances, provided that the specimens are cylindrical and fit the dies with the same clearance that a pin usually fits in the hole for which it is intended.

It is evident from the results of Mr. Hunt's shearing tests, that the double shear would give more accurate results than single, as it comes more nearly to eliminating the frictional resistance than the case of single shear.

T. S. White, M. Am. Soc. C. E.—One question I would like to ask Captain Hunt, in this method which Mr. Morison aptly styles an ideal method of testing if it could be applied in shop practice, whether the time required in punching is an essential? Captain Hunt, in one place, speaks of the time required being from 30 to 60 seconds for each test.

JAMES CHRISTIE, M. Am. Soc. C. E.—The method of testing structural steel proposed by Mr. Hunt offers a convenient and ready means for testing material in quantity, without submitting to the expense and delay of preparing specimens as now practiced for tension tests. If the upper shank of the punching tool was formed into a tapered drift, and a segmental die used which would open laterally when penetrated by the drift, the operator could not only measure the resistance to or work done by punching, but also that due to the distension of the specimen by the drift.

This is a test of great value for ascertaining the quality of iron or steel. It is a fact well known to inspectors, that sometimes steel will be found that does not properly endure the torture of the punch and drift, or bending test after punching, and yet yields satisfactory results when pulled in the testing machine in the ordinary manner. Nor is the chemist always able to disclose the secret of a particular weakness, which leads some to declare that no steel should be punched or sheared. It is quite probable that if specimens were sheared to a definite dimension, and offered a definite resistance to punching and drifting, and the punched holes distended a definite amount before cracking, we would have as useful a criterion as could be devised for judging the quality of structural steel.

A. C. CUNNINGHAM, Assoc. M. Am. Soc. C. E.—In a discussion upon steel rails before the American Institute of Mining Engineers in February, 1881, William Sellers, Esq., of Philadelphia, proposed the use of a registering punch to be used on the cross end of rails, as one of the methods for determining their quality. Mr. Sellers calls attention to the difficulty of applying a registering apparatus to regular punches, and concludes that, if used, a special machine should be adopted.

In the same discussion Professor Egleston, of New York, states as follows :

"Mr. Sandberg in his paper mentions the idea of using a registering dynamometer attached to the punching machines, and of determining the quality of the metal by the resistance which is shown. I think the first idea of this kind was published by Professor Langley, of Pittsburgh. * * *"

We thus see that the idea of determining the quality of steel by its resistance to punching has been considered before, but it does not seem to have met with much favor, as it has not come into general use.

The present methods of testing, which Mr. Hunt proposes to

abandon for a method of punching, shearing or drifting, have been used for many years, and are the development and result of observation and study on the part of those who manufacture and work in steel. Each test has its specific purpose and tends to develop certain qualities of the steel.

The tension test grades the metal, and the appearance of its fracture is a valuable guide to its properties, the combined results giving valuable indications of lack of work, cold rolling, overheating and burning.

Hot, cold and quenched bending tests give reliable indications of the presence of excessive amounts of the various constituents of the steel, as carbon, phosphorus, sulphur, manganese, etc., which are not developed by the tension tests. Drifting tests are not commonly made, but are of value in developing the ductility of the steel. Drop tests are of decided value for material subject to impact, as car wheels, rails and axles.

Other physical tests not mentioned by Mr. Hunt, but often specified or used, are as follows :

Drawing down the steel a certain percentage of its thickness under the hammer, either hot or cold, and called hot and cold hammer tests. Special punching tests to demonstrate how near together, or how close to a sheared or rolled edge, holes may be punched. Opening and closing tests in which the different parts of a shape, as the legs of an angle, are closed together or opened out. Upsetting tests in which the steel is driven back on itself. Nicked bending tests for the comparison of different steels, or the homogeneity of the same steel.

The use for which a steel is intended, or the treatment it is to receive, will develop suitable tests to be employed.

The chemical analysis of the steel, the most important of all, will in itself indicate to the practical observer the ultimate strength of the metal, and whether it is liable to hot or cold shortness, sensitive to impact or otherwise likely to prove dangerous under certain conditions.

The use of all the above-enumerated tests is not necessary for determining the value of a steel for structural purposes, and it is rarely the case that they are all applied. With the analysis of the steel, a tension test, a cold and quenched bending test, no steel need be accepted for a structure that is not entirely suitable; and if these tests are properly selected, one of each is sufficient for each cast of steel.

By cutting the ordinary tension test from 6 to 8 ins. longer than usual, the ends of the broken pieces can be used for making the bends, and the actual number of machined pieces required for three tests is thus reduced to one.

These methods of testing steel Mr. Hunt proposes to abandon for the following reasons :

First.—On account of the time required and expense occasioned, the tensile, chemical and drop tests are not sufficiently numerous to discover and check a lack of homogeneity in the metal.

In this connection it may be said that the time required to prepare an ordinary tension test will not exceed two hours from the time the metal is cold enough to cut ; the number that can be actually prepared depending upon the facilities for so doing. In many mills 100 tests can be averaged daily. In one of the mills in this country the cost of preparing tests has been carefully computed and found to be about 22 cents per test.

The analysis of some of the elements of the steel can be determined before the metal is cast. The analysis of all the elements can be determined before the steel is ready for rolling. Drop tests can be made on the cross-ends of the material and require no preparation whatever. Any delays on the preparations of tests or analyses may be traced to lack of proper facilities for making them.

Second.—Variation in results of tests owing to location from which they are taken, and variations due to speed of testing, amount of impact, application of strains, and by pulling specimens out of line.

To these objections it may be said that the variations of results due to location of test piece will apply equally as well to a punching test as to a tension test. The amount of impact is settled by the specification for drop tests, and the speed of testing and application of strains cannot make a poor steel give good results. Pulling a specimen out of line is against the development of best results, and cannot be used to pass questionable steel. Location of test piece, speed of testing, etc., are all within the control of specifications.

Third.—The narrowing of the allowable limits of specifications by engineers, and the strict interpretation of the same by inspectors.

In this connection it may be said that the bridge builder's standard specification, made by the contractors themselves, set the allowable variations for the ultimate strength of steel at 8 000 lbs., and it is the

exception to find a specification in which the limits are narrower than this; where they are narrower, it is because the engineer wishes a still more uniform metal, or has been led to believe from his test reports that a more uniform metal can readily be made.

The specification of the city of Chicago allows a variation of 13 000 lbs. in finished material, and the specification of the U. S. Navy Department for the structural material of the new ships calls for a minimum ultimate of 60 000 lbs. and a minimum elongation of 25% in 8 ins., with no maximum limit of ultimate and no requirements of elastic limit or reduction of area.

Certainly, the narrowing of some specifications seems to be more than offset by the widening of others. No contractor is obliged to make material under a specification that he considers too narrow, and, if in doing so he has more or less steel rejected, he should find the fault in himself, and not in the engineer who made the specification, or the inspector who enforced it. Inspectors do not contract the requirements for steel inside the specifications; the manufacturer may be depended upon to enforce this, and engineers have doubtless found out that it is unwise to give inspectors unlimited powers to accept steel at their discretion.

How the substitution of a punching test for a tension test is going to remedy the narrowing of specifications, unless by temporarily befogging those who make specifications, is not readily apparent. The engineer who requires close limits for a tension test will require the same for a punching test, when he becomes familiar with it.

Fourth.—"Bending, drifting and similar tests of ductility do not give results, in numerical quantities, convenient for reference, comparison and record, or for exact stipulation of requirements in specifications."

Specifications give the angle to which tests must be bent, and the diameter of the circle around which they are to be bent. The angle and curve, whether greater or less than required by specification, to which tests conform, can be recorded numerically, or graphically, by placing on paper and drawing pencil marks around them. In the same manner, in the case of drift tests, the diameter of original hole and of drifted hole can be recorded numerically or graphically. In fact, any test involving a change in dimensions, or alteration of figure, can be recorded numerically.

The method for testing steel which Mr. Hunt proposes to substitute for the present method consists of registering the results of punching holes in the steel; the same cross-ends from which we take our present tests are to be utilized for the punching tests, and owing to the rapidity with which the tests can be made, and the slight amount of preparation necessary, a much greater number are to be made on each heat of steel.

The claims for this method do not hold in the case of car axles, heavy eye-bar flats, large rounds for bridge pins, rounds and squares of any dimensions, flanges of beams or channels, or any material of excessive thickness, or having surfaces not parallel. In such cases, fully as much, if not more, preliminary preparation will be required to prepare the tests than is now the case.

It is further proposed to equip the shop punches with a registering apparatus, whereby every hole punched in a piece of material may become a test.

Besides the preliminary cost of equipping the punches, it would seem that considerable delay in the work would be occasioned, even by recording the results on a part of the holes, and a more accurate observer than the man running the punch would be needed to make the observations.

The argument has been used that the bridge shops are subjected to annoying delays by the present method of testing, but by the proposed method it would seem that they might be subject to even greater delays and uncertainties.

In the case of drilled work, the shop observations become impossible.

The unavoidable variations in the conditions of punches and dies, and amount of clearance between the same, would alone seem to offer an insurmountable obstacle to this method of testing.

The punching tests made by Mr. Hunt were conducted under uniform conditions, but the results are so variable as to be rather discouraging in themselves, and, when the further variation of conditions is considered, it seems doubtful if this is a desirable method for testing steel.

With our present knowledge of steel, the need of detecting a lack of homogeneity is no longer necessary, for we may assume, without error, that no steel is homogeneous. What we need to determine is

that none of the steel that goes into our structures is outside the limits of safety.

In a paper by Mr. Pourcel, before another branch of this Congress, the subject of segregation is very thoroughly discussed. It has been shown that, no matter how thoroughly "mixed" or homogeneous a steel may be while molten, that, as soon as it is cast and begins to solidify, its constituents begin to separate out or "segregate" towards the still molten part of the metal. This tendency to segregate is decidedly marked for carbon, phosphorus and sulphur, and less marked for manganese. In consequence of this law of segregation, it is found that the elements of the steel tend to accumulate towards the axis and top of an ingot as long as any of the steel is molten, and will be in excess at the top near or on the axial line.

If the tests representing the steel are taken from the most highly segregated portions, the worst condition is at once known, and it may be assumed that the balance of the steel is better than that represented by the test. Thus the true condition of a steel may be shown to better advantage by the judicious selection of a few tests than by the indiscriminate selection of a large number.

Mr. John Purser Griffith.*—Mr. President, the group of interesting papers on steel which the Civil Engineering Division of the Congress has had before it to-day proves to me that American engineers, like their European brethren, are rapidly appreciating the value of steel for structural purposes. In England, if the difficulties which lie in the way of welding steel could be overcome, there can be but little doubt that the manufacture of wrought iron would cease for commercial purposes. I should very much like to hear the opinions of some of the distinguished steel manufacturers of this country as to the practicability of welding steel with any degree of certainty. Cases occur to me in which steel bars have appeared to be perfectly welded, but, on being tested, it was proved that the surfaces in contact were only folded together or interlaced, and not welded. The uncertainty of steel welding is undoubtedly a drawback to its use for structural purposes, and I trust that the day is not far distant when this difficulty may be overcome. Mr. Hunt's suggestion to test steel by punching and measuring automatically the work done has interested me much. He no doubt is aware that the punching and drifting tests

* Of Dublin, Ireland.

are in use in England; but, as far as I know, no proposal has been made to register the work done in punching.

More than thirty years ago the late Mr. Robert Mallet proposed the adoption of a test of the work done in breaking iron by tensile stress. It appeared to me such a rational test that I have often wondered why its value has not been more fully recognized by engineers. Perhaps the absence of good automatic registering appliances, such as those described by Mr. Hunt, may explain the apathy with which Mr. Mallet's proposals were received.

ALFRED E. HUNT.—Mr. Morison states that "this proposed method of testing would, perhaps, be an ideal one, so that no other inspection would be required." It is not my judgment that this form of testing will ever replace mill inspection. One serious objection is that when the material is condemned at the bridge shop, the delays occasioned by the replacement are of such a serious character as to cause the use of material which would otherwise be condemned and which ought not to be used for structural work. Again, the tracing of steel as separate melts at the mills where it is manufactured, and reasonable supervision over this system of selection of the steel by melts is, so far as my experience goes, a necessity, to insure the proper inspection of steel.

I would not suggest the method to do away with full-size eye-bar tests. I believe this to be a very important and necessary series of tests in the proper inspection of eye-bars for any important work; but the same method of testing steel applicable for eye-bars and pins can be readily used; that is, the force required or the work done in cutting a given indenture can be measured as a method of testing with the larger blocks of steel used in eye-bars and pins.

Mr. Morison says: "I know of no method of testing eye-bars which will give perfectly satisfactory results, except that of taking every precaution to obtain uniform steel and then testing a number of the full-sized bars." I most thoroughly agree with Mr. Morison's statement in this, but respectfully submit that this method of testing can be used as one of the "precautions to obtain uniform steel," and by the ease and rapidity with which the tests can be made, more of them can be made upon any given lot of steel, and, therefore, more assurance obtained as to the uniformity of the steel in its selection for the fabrication of the eye-bars in the first instance.

While I admit that we have so far obtained more uniform and regular results with punching than with cutting and shearing, at the same time the same principle can be made to apply very satisfactorily with shearing, and I would refer any one who wants to investigate this matter further to the very valuable and exhaustive series of papers by H. V. Loss, published in the *American Engineer and Railroad Journal* about a year ago, with reference to the investigation of shearing work.

Another point, brought out by Mr. Buck, that it would be well to have the size of the punch, the thickness of the plate, proportion of size of punch to that of the die, and the sharpness of punch and die specified, or at least to eliminate as many of these varying conditions as possible. The clearance of the die should certainly be specified in any given series of tests. I think a study of the results of the tables will show the work upon the relative thickness of plate to be quite closely approaching the formula up to at least $\frac{5}{8}$ in. thickness, and the relative sharpness of the die can be readily checked by having standard pieces which take a known force to punch, or a known amount of work to punch, and checking the apparatus in this way often.

Answering Mr. White's questions, I think the appendix referring to the question of time shows that, while the conditions of the testing should be relatively as nearly alike as possible, a considerable variance in the time of making the punching test does not have a very material influence upon the results.

The suggestion made by Mr. James Christie of having the "upper shank of the punching tool formed into a tapered drift, and a segmental die used which would open laterally when penetrated by the drift," seems to me to be a very excellent one, and the data gotten by obtaining the distension of the specimen in punching or drifting would be valuable.

Answering the points of Mr. Cunningham, I wish to again call attention to the fact that it is clearly stated in the paper, and I wish to reiterate, that there is no proposal by me to abandon the present methods of testing. My idea is that the method of testing suggested can be much more readily and cheaply employed, and that it could be used as an addition; and very likely after it had been used in conjunction with tensile testing for some time it would be found to take the place of very many tensile tests. I do not believe, however, that it will ever be advisable to use this proposed method to entirely supersede the present methods of testing.

I thoroughly agree with Mr. Cunningham in his statement that drifting tests are of value in developing the ductility of steel, but I cannot agree with him in his statement that "with the analysis of steel and a tension test, and a cold and quenched bending test, no steel need be accepted for a structure that is not entirely suitable; and if these tests are properly selected, one of each is sufficient for each cast of steel." Melts of open-hearth steel are now made with from 10 to 30 tons of metal in each case; and this limited number of tests suggested by Mr. Cunningham I have found in my experience does not always record poor results, and cause the rejection of steel which is faulty on account of being non-homogeneous. This non-homogeneity is due to many different causes, among them being:

First.—Poor melting and irregular steel in its first manufacture in the steel furnace or converter.

Second.—Segregation of the impurities in large ingots of the steel.

Third.—The burning of portions of the steel.

Fourth.—The lack of sufficient work upon a portion of the metal.

The series of tests which Mr. Cunningham has outlined can often be obtained, and entirely fail to discover lack of homogeneity due to any one of these causes in a portion of the metal; the parts of the melt from which the tests had been obtained being entirely satisfactory, while other portions of the metal might be dangerously weak.

It is for this reason especially that I consider this method of tests a valuable one, as it allows more tests to be made upon any given lot of metal, giving thereby a better idea of its quality than can be obtained by the tests which Mr. Cunningham has enumerated, all of which I grant are valuable tests, and which have been so recognized and have become standard.

The cost of test specimens which have been made I grant has been increased in some of the larger mills, and the cost of preparation has been decreased by special apparatus. At the same time "100 test specimens pulled in a day," in a large steel mill are often not enough to prove the quality of the steel. Especially has this been the case when the steel works are working poorly and irregularly, and producing non-homogeneous steel. By the method of testing which has been outlined in this paper, 100 tests can be made and recorded in an hour, with a very small proportion of the expense that now is called for by the standard methods suggested by Mr. Cunningham, and which in ten-

sile tests under very favorable conditions he estimates to be about 22 cents in the preparation of each specimen.

I grant the truth of Mr. Cunningham's observation that variance in results, due to the location of test specimen, will apply equally to the punching test as to the tensile test; but this does not do away with the force of the statement given above of the great advantage of more tests, but rather adds to the necessity of the larger number of tests being obtained as can readily be done by the suggested method of measuring the force or work done in punching.

As to the substitution of punching tests for tensile tests remedying the narrowing of specifications, I have hopes that it will, for I believe that many intelligent engineers, in making the narrow limits to their specifications, do so with the idea that the narrow limit is required in order to insure homogeneity of the metal, and that, should they be able to get a much larger number of tests of any given lot of steel by this proposed method of punching, they would be willing to allow, with the larger knowledge of the character of the metal which could be obtained, a wider variance in the requirements of their specifications.

The same statements that I have made answering Mr. Morison's discussion regarding this method of testing being applicable for eye-bars and bridge pins of course would apply to car axles and large rounds and squares and other large sections of steel. It will be perfectly practicable to measure the force applied in cutting a given indenture in these large sections and thereby obtain a measure of the quality of the steel according to this method, although I have frankly to admit that this will require a considerable amount of data to be obtained by further experiments. At the same time I believe that this method can be adopted for this class of work. Of course, the method is more applicable to the very much larger number of sections of steel which are used in ordinary structural work, namely, plates, angles and ordinary structural sections which have some of their surfaces parallel.

Mr. Cunningham states that in the case of drilled work especially observations would become impossible upon this method of testing. I would call his attention to the fact that the same practice, I believe, will apply with considerable success in measuring the force and even the work done by a drilling tool as with a punch; and, from some data which I have lately obtained on this method, I am confident that a very

successful method of measuring the force required to cut a given indenture with a drill can be readily obtained.

I do not agree with Mr. Cunningham's statement that "the unavoidable variations in the conditions of punches and dies, and amount of clearance between the same, would alone seem to offer an insurmountable obstacle to this method of testing." Some of the experiments outlined in this paper, as well as a large number of others, have shown that considerable variation can be allowed in the condition of the punches and dies without materially altering the results, and that it is perfectly possible to keep the tools used in this method of testing in such condition without any extraordinary care or considerable expense, so that these considerations are not serious obstacles. However, I grant that it would be necessary in this method of testing, as in any other, to keep the condition of the punches and dies and the amount of clearance between them reasonably accurate. This can readily be done, and I again call attention to the fact that there is an easy method of checking the conditions upon standard pieces of steel which have been tested under favorable conditions.

Mr. Cunningham certainly cannot have studied the various appendices to the paper and state that the "conditions under which these punching tests were made were uniform." Many variances were obtained, both as to the thickness of the metal and character of it in being good and bad steel, and with two different sizes of punches.

I do not agree with his statement that the "results of the tables are so variable as to be discouraging" in this; and my experience does not lead me to agree with the further statement made by Mr. Cunningham that "because a test specimen may be taken from where presumably the highest segregations occur in the steel ingots, therefore the worst conditions have been taken," and that it "can be fairly assumed that the balance of the steel is better than that represented by the test specimens." I have already shown that there are other causes of non-homogeneity beyond that of segregation, and my own experience has shown that it is by no means safe to claim that any test specimen can be selected beforehand which contains the maximum content of impurities due to segregation in the steel.

Another objection has been brought to my attention, that the compressibility of the punch, as compared with that of the material, may be an important factor in this method of testing, and that I have not

mentioned this in my paper. By careful measurements of the punches used, before, during and after the punching operation, the results have shown that with ordinary work, and where the punch is not required to push through material of a thickness over the diameter of the punch, the compressibility of the punch is practically *nil* and is not a factor which enters into the measurement of the force applied, and, in fact, is not even a considerable factor in the measurement of the work done in the punching operation.

While I have to again frankly admit that there is much more work to be done upon this method of testing in order to utilize it completely, still, with each added day's experience with this method of testing, I am more convinced of its practicability and utility in testing structural material.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

681.

(Vol. XXX.—December, 1893.)

THE USE OF MILD STEEL.—DISCUSSION ON PAPER No. 643.*

By G. H. THOMSON, D. J. WHITTEMORE, GEORGE S. MORISON, ROBERT MOORE, WILLIAM METCALF and C. WEYRICH.

G. H. THOMSON, M. Am. Soc. C. E.—In Mr. Weyrich's paper, Vol. XXX, page 234, he states: "It is therefore urgently desirable that the previously approved mild steel may still continue to be used for all constructions without any alteration whatever."

The mild steel of Germany may not be, and probably is not, identically the same as the mild steels of our country; but, in view of the large quantities of so-called mild steels now thrown out on the market, much of it of a very doubtful character, it seems to me that we are justified in exercising more than usual caution in its use. We should use harder steels rather than softer steels—not too hard, however.

The main point in the quality of steel is to get ingots free from gas —a difficult matter in basic steels, unless the ingots are "cropped" for perhaps 40% of their length.

* "The Use of Basic Mild Steel as Material for Construction in Germany." By C. Weyrich, Engineer of River and Harbor Board at Hamburg, Germany. *Transactions*, Vol. XXX, page 219.

I have in mind a number of railroad bridges which have failed, or are failing, or are beginning to show evidences of failures due to the use of too soft, rotten and gaseous basic steels.

Railroad managers cannot be too wary in the acceptance of this uncertain and not very well-known material; especially should such metal be kept away from the punch and shears.

D. J. Whittemore, Past President Am. Soc. C. E.—We are given to understand by Mr. Gleim that it is the intention to use steel throughout for bridge construction in Germany. I wish to say that in conversation with two engineers of trunk lines in this country I learned that it was their intention to use mild steel in eye-bars only, and that the balance of their structures would be constructed of wrought iron.

There can be but little doubt that considerable so-called mild steel of poor quality has been manufactured and used in this country. But a short time since some girders of this so-called mild steel were constructed in Chicago which required two cars for transportation. By mistake the coupling-pin joining the cars was drawn at the time shipment was to be made, which permitted the dropping of one end of the girders to the track—a distance of 4 or 5 ft. The result of this accident was that the girders broke in two; and certainly this should not have occurred had they been built of good material. Similar failures undoubtedly have caused mild steel to be looked upon with disfavor. Proper inspection of the manufacture of the steel should be insisted upon, to overcome this prejudice.

Mr. Thomson.—I had an experience several months ago with a plate girder 140 ft. long and some 10 ft. wide. The girder was being loaded on a train of cars at one side. When up about 10 ft. the tackle failed and let one end of the girder down. The splice in the bottom flange stretched, but there was no other injury to the girder.

Geo. S. Morison, M. Am. Soc. C. E.—Speaking of injuries to bridges, I should like to mention one that occurred to a steel draw-bridge built under my direction. It was the bridge across the St. Johns River at Jacksonville, Fla. A schooner which was towed by a tug attempted to go through the draw without giving proper attention to steering. The tug in coming down the river followed a course which made a sharp angle with the axis of the bridge and, when opposite the draw, turned suddenly and went through the draw opening, expecting the schooner to follow the same course. The course of the schooner, however, was deter-

mined by the direction of the tow line. First the schooner struck one of the piers and knocked off a cathead, losing one anchor. It then rebounded, passed through the draw opening, struck the draw protection, knocked off the other cathead and lost the other anchor. It then struck one of the vertical posts of the draw with the bowsprit, the force of the blow being so great that the bowsprit was broken off where it entered the hull of the schooner, it being a solid pine stick 26 ins. square at the place of fracture. The force of the blow bent the post which was struck, bent the bottom chord, and was such as to send the draw home and close it. As the chord was bent up at the post it was down at the end, so that when the draw was closed it locked itself, and it was eight hours before it could be opened. All this was the result of a single blow. The only member that was so injured that it had to be replaced by a new member was the vertical post which received the blow, and this in my judgment was not so seriously injured as an iron member would have been in the same place. The draw was patched up, a temporary timber post was inserted and used for the remainder of the season, and in the following summer, when traffic was light, a new post was put in. The bridge was all of medium open-hearth steel, punched and reamed.

ROBERT MOORE, M. Am. Soc. C. E.—Referring to the examples that Mr. Whittemore has given, I may mention two in my own experience. In the construction of a line of elevated railway I was afraid to use steel for the anchor bolts. They were large anchor bolts, and in deference to the general opinion as to the danger of cutting threads in steel, I used iron. The only members of the whole structure, however, that failed, were two of those iron anchor bolts. If I had used steel I should have been sure that it was the fault of the metal.

On the other hand, a post in the same structure, made of mild steel, sustained the shock of a collision with loaded freight cars, without serious damage. The cars were entirely reversed in position and brought to a standstill, but the post was so little damaged that it continued to do service for over a year.

A MEMBER.—Were those screws with upset ends?

Mr. MOORE.—Yes, sir.

A MEMBER.—They had been upset and annealed?

Mr. MOORE.—They had been upset, and many of them had been annealed.

A MEMBER.—Did they break in the thread?

Mr. MOORE.—They broke in the thread, or close to the thread.

WM. METCALF, President Am. Soc. C. E.—Mr. Chairman, the only apology that I have to offer for taking up the time in this discussion is that my name has been mentioned once or twice to-day, and that I think I can point out some of the dangers to engineers in the use of steel, which will, perhaps, help them out of difficulties in regard to it.

You hear a great deal about mild steel and making steel mild by getting low tensile strength and great elongation; this is especially true in regard to boiler steel. I have only heard that kind of steel properly characterized once in this discussion, and that was by Mr. Hunt, when he said it was rotten. The matter with your mild steel, when it is too low in tensile and too great in elongation, is that it is simply rotten, by being melted, or blown, too low in carbon, because of the impossibility of removing afterwards the excess of oxygen and nitrogen that is thrown into it by that operation. After a number of years of careful study by Professor Langley, as well as in practice, we were convinced that the lowest blown steel and the lowest melted steel contained the most nitrogen and the most oxygen. We also found to our surprise, that the reason that we could sometimes put into the crucible the best material in the world and melt it into a rotten product was because we melted it too low in carbon; it would not take a bending test or a welding test. In the nice crucible work we were compelled to be cautious about melting too low. When we first found this, we kept getting it lower and lower and worse and worse, until we discovered what the trouble was. If engineers could get at this matter properly, could insure that no steel was ever melted in open hearth below 15 carbon and none blown in Bessemer below 15 carbon, and they need not be carbonized above 20 to 23, which is sufficient to remove all of the oxygen which can be removed; they might have a steel that some would call high steel, but it would be vastly better and safer than any steel of .04 to .06 carbon. If I knew it, and I were making boilers, for instance, no such steel as .04 to .06 carbon should ever go into a boiler of mine. We have numbers of boilers in our works where the carbon is up to 25 and 30. There is no danger of those boilers bursting. I have seen them bagged away down by bad firing and set up again without trouble. The secret of the failure of steel to weld is generally from too low carbon and being too high in

what we call gases. Steel really does not weld in any case, and it does not stick together properly if melted too low or blown too low. If engineers could get the idea out of their heads that the maximum safety is in the minimum strength, and would use a steel never melted or blown below 15 carbon, and which need not necessarily go above 25 carbon, I think all of these difficulties would disappear. In this exceptionally mild steel, so over mild or over blown, it is so full of gas that it is almost impossible to get a sound ingot.

Mr. Morison spoke yesterday of boring his pins on account of being piped. Sir Benjamin Baker spoke of members where the steel failed from the interior. It was not sound in the ingot; it could not be made sound. You may test it and not discover the defects, yet they are there. In hammering round pins there are liable to be holes in the center which are caused by hammering; by boring a hole through the middle of the pin you remove any possibility of there being a defect there, or, if it is there still, it can be seen and the pin can be rejected, and thus get a safer pin; because if there is a place in that pin, even at the center, where the integrity of the material is destroyed, the constant deflections will start a fracture at that defect, and it will grow until it goes through all of the material and causes a breakdown.

Mr. Whittemore.—A mild steel?.

Mr. Metcalf.—Any steel, all steel. I have only taken the time to say what I have, because I think the dangers in the use of steel have been increased by the use of too mild steel. You cannot get too much sulphur in it, because it cannot be worked. With regard to melting in the open hearth, if you do not melt too low, a certain quantity of phosphorus will come out in the process any way, and excessively low melting is not necessary. The harm done in melting away down to take out all of the phosphorus, is shown when testing; the oxygen, nitrogen and hydrogen are elements of harm which cannot be removed to a safe degree when the steel is too soft blown or melted.

Mr. C. Weyrich.—In regard to the mild steel treated of in my paper, I beg to repeat that it refers to the use for bridges and buildings, and not for engine or railway construction, such as rails, ties or rolling stock, for which purposes a somewhat harder material would be in place. There can be no question as to the milder material required for bridge building being more difficult to produce, particularly by the

Thomas process, special care being required to avoid red-shortness. This has led to the tests in Germany being made in two opposite directions, excluding too hard a material on one side, and preventing its being too soft or red-short on the other side, by specifying a lower as well as an upper limit of textile strength. For a steel tested in this manner there would be no more objection to shearing and punching than for wrought iron; and if the specifications nevertheless exclude these methods of working, the reasons are about the same as those excluding their use in wrought-iron structures of importance, a better quality and greater neatness of work being aimed at.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

682.

(Vol. XXX.—December, 1893.)

STABILITY OF QUAY WALLS.—DISCUSSION ON PAPER No. 646.*

By PETER VEDEL and C. O. GLEIM.

Mr. PETER VEDEL.—The importance of this question to all harbor engineers would excuse me for a few remarks. In its original shape this wall was of the combined type, partially a stone wall and partially a bulkhead. As such, it had to act on two quite different principles. The wall resists the pressure of the earth by its weight; the bulkhead, by the passive pressure of the earth in front of it. It does not call into action any other force; it makes the earth pressure, so to say, annihilate itself. I am well aware that this advantage is only, to a great extent, apparent, inasmuch as part of the errors in the assumptions as to the constants of the earth, instead of eliminating each other, rather, as entering into the active and passive pressures with different signs, will increase the uncertainty of the calculation. The top part of the wall is easily calculated by the usual formulas for determining the earth pressure on it. We know that, if its back side is given a certain batter, those formulas will give the same result as the rational theory of earth pressure based upon the differential equations, and we know,

* "Method Used to Secure the Stability of a Quay Wall at the Port of Altona, on the Elbe, in Germany, which had Shifted its Position after Completion." By Berthold Stahl, Stadtbaurath at Altona, Germany. *Transactions*, Vol. XXX, page 284.

too, that the pressure found will then be the actual one, whether the assumptions of one theory or the other be nearer the truth. But the bulkhead which forms the substructure of this wall is not so easily calculated unless it is decided to construct it as a usual bulkhead, to sustain the earth pressure independently of the weight of the stone wall. It seems very dangerous to count upon the joint action of those two different principles. The bulkhead may allow a certain small movement by its elasticity, just sufficient to crack the wall. The absence of land ties in the original type seems to indicate that the substructure has not been considered an independent bulkhead. In the altered wall I understand that the stone-wall principle has been abandoned altogether and the bulkhead principle adopted exclusively. True, the old front wall carries part of the weight of the cranes, but the new concrete wall which has been built behind the sheet piling takes up the pressure of the backing, and the sheet piling must, therefore, in connection with the piles, sustain the whole pressure by the passive earth pressure in front of the piles and on the wooden shields at the end of the stays.

I happen to have been in contact with a similar case, in the new harbor of La Plata, at Ensenada, in the Argentine Republic. The wall was 10 or 11 m. high, and the lower part was built of concrete deposited between two sheet pilings, which were supported by two rows of vertical piles driven into the solid clay. The top part of the wall was of masonry and fully strong enough; but, as a whole, the quay wall could not sustain the pressure of the loaded backing by its own weight, and it had to depend on the co-operation of the vertical piles. The result was that about 1 250 ft. gave way. To unload it by removing the earth and filling in with mattress work did not stop the motion. It was then tried to anchor the wall by chains to the piles, on which the superstructure of the sheds rested, but at an unusually low water the motion took greater dimensions. It was almost exclusively an overturning around the toe of the wall. The outer row of piles under the sheds followed the motion, and were nearly drawn away from under the superstructure. With other chains they were secured to the back row of piles under the sheds, but even then they began to move, and only the rise of the water prevented further destruction. In the reconstruction of this wall it was decided, instead of adopting, as in Altona, the bulkhead principle altogether, rather to do away with it; and a new,

substantial concrete wall was built in front of the old one, sufficiently strong to stand the earth pressure by its own weight, independently of the piles. It is a curious coincident that the cost of this work was about $500 per linear meter, or, considering the higher wages and transportation costs, the same as in Altona, by the reconstruction upon the opposite principle.

In the harbor of Copenhagen, Denmark, walls have been built upon the combined principle, as far as materials go, but not in the dimensioning of the construction. In renewing old bulkheads their over-water part is cut away; sheet piles and bearing piles are driven and connected with the land ties of the old bulkhead, and, where necessary, new and longer land ties are provided for; and on that substructure the top stone wall is built. In that way the under-water part of the bulkhead is the same or stronger than before, and can, independently of the weight of the wall, stand the pressure. To sum up, it seems advisable, therefore, in the dimensioning, to count only with one of the two principles acting at a time, even if other considerations make us choose to use such combined construction.

C. O. Gleim, Cor. M. Am. Soc. C. E.—In reference to the criticism on the original plan, I beg to say that the same design for the cross-section and foundation of quay walls has been employed in Hamburg, where such constructions have answered the purpose very well. I do not see how the substructure can be looked upon as a bulkhead acting on the principle of resistance by the passive pressure of the earth in front of it. I do not consider the lower part as anything but simply a pile substructure intended to take up the vertical and horizontal stresses brought upon the wall and the sheet-piling. The land ties that were afterwards constructed were not originally required to make up the stability of the quay wall; nor would they have sufficed afterwards unless they were fixed in the stratum of clay underneath. According to the author's representation this was a case of a landslide taking place along a certain sliding surface, as shown in Fig. 1 on the plate, and carrying away the quay wall together with its back-filling and the freight shed. The wall has now been anchored to the fixed subsoil. No anchorage constructed on the principle of an ordinary bulkhead anchorage, even with sufficient weight of the ground resisting the anchorage, would have answered the purpose if the anchors had been placed in the sand-filling over the sliding surface.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

683.

(Vol. XXX.—December, 1893.)

UNDERGROUND SOURCES OF WATER SUPPLY.—DISCUSSION ON PAPER No. 647.*

By CHARLES B. BRUSH, L. L. TRIBUS, D. J. WHITTEMORE, D. W. MEAD and B. SALBACH.

CHARLES B. BRUSH, Vice-President Am. Soc. C. E.—I ask your attention to the paper by Mr. Salbach, of Germany, which has many points of interest; it is a discussion of one of the most important questions that comes before us, viz., the question of subterranean water supply. The author dwells at length on the fact that large subterranean supplies are always obtained in valleys. He says:

"The reason for the location in valleys of water works depending on a supply from wells is generally to be found in the situation of gravel and shingle deposits which, reaching down from the mountain ridges to the valleys and carrying off spring water, are exposed to a greater extent in the deep cuts formed by the river valleys than elsewhere. The greater part of these depressions was formed by movements upon the earth's surface a long time before the existence of rivers as they are now, and the deposits of gravel and shingle referred to are of much more ancient origin than the rivers themselves, which latter are generally excavated in more recent formations. Especially inter-

* "Experiences Had During the Last 20 Years with Water Works Having an Underground Source of Supply." By B. Salbach, Baurath at Dresden, Saxony, Germany. *Transactions*, Vol. XXX, page 293.

esting examples in this regard are the sources of supply of the water works in the valley of the Rhine, of Bonn, Cologne, Elberfeld, Düsseldorf, Mannheim, and also of the city of Dresden."

After discussing this point at considerable length, the writer goes on to show that the prejudice that exists against subterranean supplies is based on the theory that water thus obtained is merely water from an adjoining river.

He says :

"Examples of the yield of large quantities of spring water for many years will counteract the often-expressed prejudice that all such works yield nothing but filtered river water, especially as the votaries of this opinion generally deny the possibility of a natural filtration of river water. The direction of a current of spring water into wells must primarily be considered, in order to get a clear idea of how the wells are fed."

Then on page 314, he says :

"Some borings put down in the bed of the Elbe itself to a depth of 23 to 26 ft. showed that the ground-water in these bore holes would rise some 6 ins. above the level of the river. An examination of the samples of the water taken from the last-named bore holes showed a quality of water which, like the samples taken on the right bank, was softer than the Elbe water and of the same constituency as that from the bore holes first named."

It is the experience of every one who has examined this question of subterranean supply that there is a great distinction between water from a river, and water taken from adjoining subterranean sources not connected with the river. A very interesting illustration of that fact may be seen at the water works at Princeton, N. J. The water supply there is taken from the valley of Stony Creek. I drove a tube well about 35 ft. deep through the bed of this creek. For several years a stream of water flowed freely and probably still is flowing through this tube to an elevation about 3 ft. above the water in the creek. The water in the creek is polluted. This water that rises in the tube through the waters of Stony Creek is pure and clear and shows by analysis to be a different water from the water of the creek itself.

In Jamestown, N. Y., there is a remarkable subterranean supply, which is undoubtedly obtained from higher lakes at a considerable distance from the town. The water rises in tube wells near the town to a height of about 40 ft. above the surface of the ground. Lake Chau-

tauqua at Jamestown is lower than the elevation at which that water rises in these tube wells.

Long Island, N. Y., presents probably the greatest example that we have of a subterranean supply in the eastern part of the United States. Wells will flow at an elevation of several feet above the sea level. The inclination of the ground-water as it rises from the sea is about 9 ins. in 1 mile. Water can be obtained in large quantities almost anywhere on Long Island by boring for it. The total amount of water supplied to the city of Brooklyn is in the neighborhood of 68 000 000 galls. per day, of which amount 20 000 000 galls. are obtained from driven wells. All along the island and in the vicinity of these city wells there are other wells, which supply large quantities of water.

Perhaps the best illustration in the western part of this country of a subterranean supply, is the city of Memphis, where they are using about 10 000 000 galls. per day. The supply is obtained from a stratum of fine sand about 700 ft. thick commencing about 200 ft. below the surface. Above the sand there is a tight, impervious material; but when that material is tapped, the water rises to an elevation of about 25 ft. higher than the water in the Mississippi River at Memphis.

A similar and perhaps the same subterranean stratum that supplies Memphis has recently been tapped at Savannah, Ga.

This paper of Mr. Salbach is interesting, not only in the particular to which I have called your attention, but in the general discussion of the whole subject of subterranean water supply. There is one phase of the paper which will be interesting to American engineers, which reads:

"The experience of the year 1892 has shown that great caution should be exercised in determining upon the actual amount of consumption. The long-continued drought of that year caused an excessively large use of water and but few water works were in a position to meet the same. It was found that the maximum consumption in that year in Dresden was over 40 galls. per day per head and the water works had to restrict the amount supplied to the public fountains, so as to provide for the event that the consumption would increase still further."

In view of the fact that many cities of this country are using 200 galls. per head per day, and but few are using less than 100 galls., it is refreshing to learn that in some parts of the world they consider 40 galls. per day per head a maximum quantity.

There is one feature which I had hoped to see mentioned in this paper which has not been incorporated; the author has not shown us how he has obtained water from subterranean sources. He speaks of putting down wells, but does not say how he puts them down. He gives no idea of the mechanical appliances used to obtain water. The natural way where the water can be obtained within a depth of not exceeding, say, 50 ft. from the surface is to sink a large open well. But it has been found that a large open well will only yield a limited supply of water. The experience of Brooklyn is that an open well 50 ft. in diameter and sunk into about 20 ft. of water has an extreme yield of only about 1 000 000 galls. per day. This, of course, is not sufficient to meet a large consumption, and therefore other means of obtaining the water are necessary. The most common is to put down tube wells and to connect these tube wells with the suction main.

For some time the principle of the ordinary driven well was employed; that is the way it is done in Brooklyn. Two-inch or 3-inch wells with screened, perforated points were driven down into the water-bearing strata, and then suction was applied directly to these tubes.

Another method is to sink a perforated casing, put a tube inside and draw from this interior tube. That is the way more usually employed now in the collection of water; that is the way that it is done in Memphis and in many other portions of the country. It would be exceedingly interesting if we were informed of the methods employed in Europe in the different towns mentioned in this paper. The supplies that have been obtained in the instances recorded in the paper are usually comparatively small. Definite statements are not made in many cases as to the amount of water supplied. Some data are given, however. The city of Bernburg on the Saale has a supply of 1 000 000 galls. per day. The supply of Crefeld on the Rhine is stated as 7 500 000 galls. per day. The supply of Dresden, of which a very interesting description is given, has reached about 12 000 000 galls. per day. The number of inhabitants is about 300 000. This city recognized the fact that it is dangerously near its limit, and it is making arrangements for an extension of the supply up to 22 000 000 galls. per day. This is the largest supply that Mr. Salbach refers to as being in actual operation. It is stated, however, that arrangements are now being made for a supply from subterranean sources for Vienna, the minimum yield of which will be 50 000 000 galls. per day, and the

maximum 100 000 000 galls. That is so enormously in excess of any subterranean supply that has been obtained in this country or elsewhere that it would be interesting if we knew the method by which they propose to obtain so large a quantity, and what the result is likely to be, especially in the surrounding country, of continually pumping so large a supply of water from a subterranean source.

L. L. Tribus, Assoc. M. Am. Soc. C. E.—Mr. Brush has mentioned two interesting districts in the South for underground water supplies, viz., those around Memphis, Tenn., and Savannah, Ga. They are separated most completely, however, by the southern prolongation of the eastern ranges of mountains. The Savannah district being more of sand, clay and ocean deposits, while the Memphis district is on the edge of the great silt deposit forming Louisiana, Mississippi, southern Alabama and western Florida.

A few years ago I had occasion to examine various portions of this latter area for underground supplies of water, and was exceedingly interested in its study. Numerous driven wells have been sunk in various portions of the tract, with excellent success in some, fair in others, and complete failure in still other parts. The water-bearing strata were found at depths ranging from 50 to 1 000 ft.

In boring these wells but little is encountered except alternating layers of vari-colored clays, sand, silt, fine gravel, and occasionally a thin layer of hard pan—a few of the clay strata being quite thick, from 10 to 50 or more feet, of a dark blue color and a fine texture, almost free from grit.

In the western portion of the tract silt predominates, being the detritus carried down by the Mississippi River from the alluvial plains of the north and west, while the eastern portion is composed more of sand, coming from the nearer mountain ranges, and, owing to its shorter water carriage, is less ground up.

Western Florida seems to be an ideal locality for the driven well—largely of this sand and gravel deposit, with little silt or vegetable matter in its make up, and the mountains of northern Alabama and Tennessee to catch the rains and start the waters southward to the Gulf of Mexico. For an area of about 100 sq. miles in this western end of Florida, the water flows freely when these gravel strata are tapped at depths ranging from 50 to 300 ft., is of excellent soft quality and about 70° in temperature. I cannot say but that greater depths

would have given increased flow, but those mentioned gave sufficient satisfaction, with moderate expense in driving.

About 10 or 15 miles out from the Florida shore in the Gulf of Mexico, and for a stretch of about 100 miles or so, there have been frequently noted vast springs of fresh water boiling up, from a depth of from 100 to 300 or more feet. This fact, with that of a rise and fall of tide of about 12 or 15 ins., only once a day, should be noted.

Back about 1½ miles from the shore front in the city of Pensacola, were driven, some seven years ago, a series of 4-in. and 6-in. pipe wells to depths from 60 to 280 ft., at a point where the ground elevation was about 20 ft. above tide level. Water rises in these wells to within 3 or 4 ft. of the surface of the ground, or to 16 or 17 ft. above tide level, and they furnish with direct suction from 50 000 to 150 000 galls. per day each, yielding a total at that time (1886) of from 1 500 000 to 2 000 000 galls. per day.

These wells, though flowing at this height above tide, rise and fall from 6 to 10 ins. daily, apparently with the tide. This looks decidedly as if they have direct free communication with salt water and are merely tapping the underground rivers which supply the great Gulf springs miles away.

All along this portion of the Gulf Coast numerous fresh-water springs are found on the beach just above high water-mark.

The strata from which the Pensacola water is drawn are almost entirely a white gravel mixed with fine white sand and overlaid with beds of a very hard blue clay, but there are many small layers of various kinds of clay and colored sands encountered in the borings.

To the eastward of this "sand," as distinguished from the western "silt," basin, which by the way is very deficient of good potable underground waters, the character of soil changes and soon reaches the limestone, cavernous spurs of the mountains, which run out beneath the surface in the peninsula of Florida.

This locality, furnishing abundant supplies of water, often under considerable pressure, for instance, St. Augustine and Jacksonville, might be called the "artesian" as compared with the "driven well" district to the westward. But these waters are universally impregnated strongly with carbonates and sulphur, and are therefore unsatisfactory for domestic use.

Following the Atlantic Coast northward the sand deposits are again

reached, yielding good potable waters, as evidenced at Savannah, Charleston, etc., as before mentioned.

These items can scarcely be considered so much discussion of Mr. Salbach's paper as additional notes on the general topic of underground supplies, from the experience of another observer.

D. J. Whittemore, Past President Am. Soc. C. E.—Mr. President, I regret very much that there has not been some paper upon the artesian features of our country. Allusion has been made to the southern portion of the country—I wish to call the attention of the members to the artesian features of Dakota. Take from Yankton, north, or the James River Valley throughout almost its entire length, and boring almost anywhere in that valley and close up to the coteaus on the other side, the valley is about 30 to 50 miles wide, an immense flow of water is struck; and so far as my own knowledge extends, the static pressure is about 50 lbs. per square inch. Those who know the features of the country, how subject it is to drought, can appreciate something of the value of an artesian supply. I trust that some member, at our next convention, will take up this matter of artesian supplies in this country and give a paper on the subject.

D. W. Mead, Assoc. Am. Soc. C. E.—This paper by Mr. Salbach is of particular value to American water-works engineers on account of the wide interest which has lately been taken in subterranean supplies in this country.

In the United States, as in other countries, the most obvious sources, the surface waters, were the ones first utilized. With the increase of population these, in many instances, have become so polluted as to render them unfit for use, and many cities are therefore seeking new supplies. Where the pollution is not a present reality, but a near future possibility, the resulting dangers, as illustrated by the Plymouth, Pa., epidemic, bears such important relation to the public health as to render a change to a source free from possible danger greatly to be desired. In new work, where it is possible to secure a source beyond the possibility of pollution, or at least where the possibility of pollution is very remote, the selection of such a source is only a reasonable and necessary precaution for the preservation of the public health.

Subterranean supplies demand for their proper consideration and development a general and local geological knowledge; for only with

such knowledge can the possibility or probability of the existence of such supplies in any locality be determined.

They may in general be considered as derived from one of three sources—

First.—From the underlying stratified rock deposits.

Second.—From the drift sheet.

Third.—From the fluvial and lacustral deposits in river and lake valleys.

1st. The stratified rocks being, as a rule, laid down in ancient seas or lakes, their water-bearing strata are comparatively extensive, and the water exists, not usually in the form of underground streams, but more like extended underground lakes as extensive as the water-bearing deposit itself, and having a current usually in the direction of the dip of the strata. In some of the less porous strata, however, such as the archean rocks and the limestones, the contained water is held, not so much in the mass of the rock itself, as in cracks and fissures. The flow of water through these cracks and fissures sometimes results, especially in soluble limestone rocks, in the formation of caverns through which the waters undoubtedly flow as streams. This is not common, however, and the ordinary idea of underground streams is largely erroneous. The water in these strata is received from the rainfall and flow of streams at and over its outcrop, and flows thence toward the sea, frequently reaching the surface, naturally through fissures in the overlying rocks, as springs, or artificially through bore holes as artesian wells. The outward pressure and depth of these waters are often so great as to preclude the possibility of any pollution before reaching the outlet, and if properly cared for at the surface, they are organically the purest of all waters, although frequently highly mineralized. Such water-bearing strata are found in the upper Mississippi Valley, in the Potsdam and St. Peter sandstones, from which numerous private and public supplies are obtained, many wells being drilled into the latter, and a few into the former, in and about the city of Chicago at depths ranging from 700 to 2 700 ft. In the Dakotas and northern Nebraska, similar supplies are encountered in the Dakota sandstone at similar depths, while the cretaceous deposits furnish the source from which Memphis, Tenn., is supplied, and from which many wells along the Atlantic coast are derived. Other similar sources exist, some utilized, and some unutilized, in various parts of the United States.

2d. The drift sheet which extends over the northern third of the United States and over the greater part of British America, is a deposit so irregular in formation that a careful study of its origin and method of deposition is necessary for a thorough understanding of its possible value as a source of supply. This deposit is the result of ice and water action, and in many cases contains water-bearing sands and gravels, sometimes of vast extent. They are, however, commonly more local in character than in the case of the stratified rock deposits previously considered.

The contained waters, therefore, are in this case more of the character of underground streams, and vary largely in breadth and depth, following the sands and gravels of ancient glacial flood deposits. Often they occur in seemingly isolated streaks and veins in thin or narrow deposits of sand or gravel, surrounded by the glacial clays, in which cases adjacent wells may differ greatly in depth and entirely in the immediate source from which their waters are derived. The general direction of flow is, as a rule, in the direction of the glacial motion, which caused the deposits. Supplies from this source, especially when near the surface, require careful observation, to determine their source, character, amount and liability to local pollution. In many cases, however, the deeper waters from the drift are organically almost as pure as the waters of the lower stratified rocks, and often contain much less mineral matter. The drift formation varies from nothing to a maximum depth of perhaps 400 ft. From this source are derived most of the common wells in the northern States, and many of the municipal supplies.

3d. The stratified deposits in old lake and river valleys which have been filled to some extent above the bed of the ancient stream or lake, offer sources more or less extensive. From such sources are derived the water supply of Peoria, Ill., and the numerous artesian wells in the Red River Valley in Minnesota. A table submitted herewith gives some data concerning some of the larger cities and towns of the United States having underground sources of water supply.

The liability of wells or galleries adjacent to streams to receive water by infiltration from the streams varies with the nature of the deposits through which the stream flows. Within the drift belt the rivers, as a rule, carry so much silt that any outflow is at once checked by the silting of the pores in its bed. The tendency, when the adja-

Some of the Larger Cities and Towns of the United States Having Subterranean Sources of Water Supply.

City.	State.	Population. 1890.	Source.	Geological Source.
Anniston	Alabama	9 998	Springs	
Appleton	Wisconsin	11 869	3 artesian wells	St. Peter sandstone.
Akron	Ohio	27 601	Artesian wells	
Atlantic City	New Jersey	13 055	Large well	
Aurora	Illinois	19 688	2 artesian wells	From St. Peter and Potsdam sandstone.
Beloit	Wisconsin	6 315	Large well	From Trenton rock.
Big Rapids	Michigan	5 303	Drive wells	From drift.
Brooklyn	New York	806 343	Drive wells	From drift.
Brunswick	Georgia	8 459	Artesian wells	
Cedar Rapids	Iowa	18 020	Artesian wells	{From Potsdam Sandstone. {River for fire service.
Charlestown	South Carolina	50 000	3 artesian wells	
Clinton	Iowa	13 619	4 artesian wells	From Potsdam and St. Peter sandstone.
Columbus	Ohio	88 150	Filter gallery	From drift.
Dayton	Ohio	61 220	65 8-in. tube wells	From drift.
Des Moines	Iowa	50 093	Large well	
Dixon	Illinois	5 161	2 artesian wells	From Potsdam sandstone.
Dubuque	Iowa	30 311	Artesian wells	From Potsdam sandstone.
Fond du Lac	Wisconsin	12 024	5 artesian wells	From St. Peter sandstone and drift.
Freeport	Illinois	10 189	18 drive wells	From drift.
Fresno	California	10 818	10 Artesian wells	200 to 600 ft. deep.
Galena	Illinois	5 535	Artesian wells	From Potsdam sandstone.
Galesburg	Illinois	15 264	Drive wells	From drift.
Grand Island	Nebraska	7 536	Drive wells	Platte River Valley.
Jamestown	New York	16 038	Artesian wells	
Jackson	Michigan	20 798	Artesian wells	
Joliet	Illinois	23 264	Drive and artesian wells	From drift and St. Peter sandstone.
Jacksonville	Florida	17 201	4 artesian wells	From cretaceous strata.
Kearney	Nebrsska	8 074	Drive wells	Platte River Valley.
Kalamazoo	Michigan	17 853	2 large wells	From drift.
Lincoln	Nebraska	58 000	2 large wells	From drift.
Macon	Georgia	22 746	Springs	
Memphis	Tennessee	64 495	Artesian wells	From cretaceous strata.
Mansfield	Ohio	13 473	16 wells, 2 springs	From drift.
Montgomery	Alabama	21 883	Artesian wells	
Michigan City	Indiana	10 776	Springs	From drift.
Muncie	Indiana	11 345	Artesian wells	
Manistee	Michigan	12 812	50 drive wells, 1 large well	From drift.
New Brighton	New York	16 423	76 drive wells	From drift.
Nashville	Tennessee	76 168	Underflow	Cumberland River, from filter gallery.
Oshkosh	Wisconsin	22 836	Artesian well	Lake Winnebago, for fire protection.
Peoria	Illinois	41 024	Large well	From gravel in ancient bed Illinois River.
Pine Bluff	Arkansas	9 952	Drive wells	
Pensacola	Florida	11 750	16 drive wells	
Pittsburgh	Kansas	6 697	Artesian wells	
Rockford	Illinois	23 584	9 artesian wells	From Potsdam and St. Peter sandstone.
Savannah	Georgia	43 186	Artesian wells	
Stockton	California	14 424	Artesian well	
Selma	Alabama	7 622	3 artesian wells	
South Bend	Indiana	21 819	32 artesian wells	From drift.
Springfield	Illinois	24 963	Well and filter gallery	From drift.
Sioux City	Iowa	45 000	Drive wells	
Winona	Minnesota	18 208	2 large wells	Mississippi underflow.

cent ground-water is low, is for the water of the stream to flow into the ground, and the reverse when the ground-water is high. This has undoubtedly resulted in making the beds of streams within the drift area practically impervious, and little permanent infiltration can be expected into wells or filter galleries from such sources. From this condition has resulted the failure of such wells and galleries as have been sunk near streams, with the expectation of obtaining filtered river water.

Outside of the drift area, however, the streams are often more free from silt, and flow in beds of sand. In such cases the visible stream is but a portion of the invisible stream, flowing under and on both sides parallel to it. Such a stream is the Platte River of Colorado and Nebraska, and during dry weather, when the visible Platte shrinks to little or nothing, water can be obtained by digging into its sandy bed. It is, of course, obvious that a filter gallery or well near such a stream will receive water filtered through the adjacent banks. Nashville, Tenn., obtains its supply from a filter gallery in a sandy island in the Cumberland River.

The seepage of water into and from the adjoining ground is shown in a series of gaugings of the South Platte River, in Colorado, a record of which from the report of the State Engineer is submitted on page 700.

The dependence or independence of a supply on the conditions of rainfall on its water-shed depends entirely on the nature and extent of such water-shed. Where the water-shed is extended and abounds in lakes, swamps and rivers, little or no change will be noticed at different seasons. The Potsdam sandstone, for example, has an outcrop in the upper Mississippi valley of about 14 000 sq. miles, over which flow numerous rivers. Hence supplies derived from this source feel comparatively little influence of temporary conditions of rainfall. On the other hand, with limited water-shed the influence is immediate; as an example of this may be mentioned the artesian wells at Denver, Colo. Mr. Wadsworth, of Arvada, Colo., says of these wells (see Preliminary Report of Artesian Well Investigation, Department of Agriculture, 1890, page 180): "Our artesian wells come and go with the seasons. When the little streams in the foothills are pouring their contents into the loose rocks * * * artesian wells in Denver and vicinity work all right; but when the streams fail in the foothills, some of the wells cease to flow."

TABLE OF MEASUREMENTS OF SEEPAGE WATER IN THE SOUTH PLATTE RIVER, COL., OCTOBER 23D TO NOVEMBER 5TH, 1891.

From Sixth Biennial Report, State Engineer of Colorado.—In Feet per Second.

Names of Streams and Ditches where measurements were taken.	Amount of water in river.	Amount of water diverted from river by irrigation ditches and canals. From last station.	Amount of inflow from natural tributaries.	Amount of water in river at points measured, plus that diverted by canals and minus the inflow from natural tributaries.	Amount of increase in volume of river between points measured.	Decrease in volume of river between points measured.	Miles between points measured.	Amount of increase per mile between points measured.	REMARKS.
South Platte River	204.35								Above dam of High Line Canal.
" "	46.16	185.76		231.92	27.57		6	4.93	Below City Ditch.
Plum Creek			2.98						
South Platte River	84.86	16.89		284.53	53.61		6	8.93	At Littleton.
Bear Creek			7.26						
Cherry Creek			5.59						
South Platte River	90.05	23.86		300.73	16.20		10	1.62	At foot Sixteenth Street, Denver.
Clear Creek			1.04						
South Platte River	79.58	53.98		343.20	42.47		11	3.86	Below Fulton Ditch.
" "	73.96	41.96		379.54	36.34		7	5.19	Below Brighton Ditch.
" "	90.28	27.18		423.04	43.50		9	4.83	Below Evans No. 2 Ditch.
" "	11.09	87.43		431.23	8.14		7	1.16	Below Farm's Ind. Ditch.
" "	17.48			437.67	6.39		5	1.29	Above St. Vrain Creek.
St. Vrain Creek			28.94						
Big Thompson Creek			12.85						
South Platte River	87.68	37.48		503.56	65.89		11.5	5.73	Below Latham Ditch.
" "	114.60			530.48	26.92		6	4.49	Above Cache la Poudre River.
Cache la Poudre River			61.11						
South Platte River	244.33			607.01	76.53		9	8.50	Below Harding Ditch.
" "	211.69	49.25		623.21	16.20		11	1.47	Below Putnam Ditch.
" "	106.69	120.28		638.40	15.29		14	1.09	Below Ft. Morgan Canal.
" "	134.81	10.14		676.76	38.36		9	4.37	Below Platte and Beaver Ditch.
" "	186.79	46.21		774.95	98.19		14.5	6.77	Above Smith Ditch at Snyder.
" "	46.68	119.84		754.68		20.27	17.5	Loss 1.16	Above Merino.
" "	66.73	13.31		788.04	33.36		13	2.59	At Sterling.
" "	52.72	42.08		816.11	28.07		9	3.12	At Iliff.
" "	39.65			803.04		13.07	15	Loss 0.87	Two miles above Crook.
" "	47.70			811.09	8.05		20	3.42	Below Sedgwick.
" "	42.96			806.35		4.74	16	Loss 0.29	At Julesburg. Average increase per mile = 3.24.

B. Salbach.—In writing this paper it was not my intention to give a technical description of the mechanical appliances used for obtaining potable water from subterranean sources, but only to show and explain the existence of underground sources and their subterranean courses. The different methods employed for obtaining the water from these sources depend on the geological formation and the extent of the subterranean water-bearing strata. In the case of thin and small pervious strata horizontal collecting galleries are used, while larger masses of gravel and debris filling the subterranean valleys are opened for supply by putting down wells. Where the water-bearing soil shows a very compact stratification, it is advisable to put down a great number of wells of small diameter—150 to 200 mm. (6 to 8 ins.), while a loose and very pervious character of the subterranean strata calls for wells of larger diameter—5 to 7 m. (16 to 23 ft.).

On this subject, as well as the results attained in the new waterworks of the cities of Vienna and Dresden, there are publications to be found in technical journals. In the meantime I am gratified to find that my paper has given rise to so instructive a discussion.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

684.

(Vol. XXX.—December, 1893.)

PURIFICATION OF SEWAGE AND OF WATER BY FILTRATION.—DISCUSSION ON PAPER No. 650.*

By EDWARD FLAD, ALLEN HAZEN, RUDOLPH HERING, THOS. M. DROWN, G. H. BENZENBERG, ROBERT MOORE and E. C. JORDAN.

EDWARD FLAD, M. Am. Soc. C. E.—I would like to ask a question in connection with the paper of Mr. Mills on purification of sewage. I presume Mr. Hazen can set me right. Some months ago, I had occasion to look over these reports. In the reports covering the purification of sewage through gravel beds, I found that at the end of the third year the gravel was unchanged, and that they succeeded in filtering about 100 000 galls. per acre in 24 hours. It was stated that filtration could be carried on for an indefinite period without the necessity of cleaning the gravel. I went on, and the next year, I think the fourth year of the experiments, the gravel bed was found to be slimy, and it was evident that it would be necessary to remove a portion of the gravel. My reading of those reports does not quite agree with the extracts from Mr. Mills' paper.

ALLEN HAZEN, Assoc. Am. Soc. C. E.—The paper states that there is a certain small percentage of nitrogenous matter in sewage which is

* "Purification of Sewage and of Water by Filtration." By Hiram F. Mills, A. M., C. E. *Transactions*, Vol. XXX, page 350.

stored in sand in any case; Mr. Mills gives it as 15% of the total amount. In the case of sand, this is stored in the surface layer where it can be seen, but in the case of large gravel stones the movement of the sewage over the stones carries this material below the surface to a point where the current is so slow that it has a chance to deposit itself upon the stones. There is no surface indication of what has accumulated. At the end of a certain time this material had accumulated to such an extent that it prevented the circulation of air through the slime; oxidation was prevented, and we got simply a straining.

Mr. FLAD.—I would like to have an explanation in regard to the direct statement which was made at the end of the third year, that the gravel could be used indefinitely for a number of years, while I thought that a directly contrary statement was made in the next report, and I was much disappointed. I would like to know what the final result is.

Mr. HAZEN.—The result? At the end of the time that you mention there was no appearance of clogging; the material, so far as we could get at it, was entirely clean; we did not know of the accumulation below the surface.

Mr. FLAD.—The extracts which were read a few minutes ago from Mr. Mills' paper did not refer to any such clogging. Upon referring, however, to his paper, which I had not previously had an opportunity of reading, I see that he fully explains the matter. The fact that the conclusions at the end of the fourth year should directly contradict those given out the year before is somewhat discouraging, and naturally gives rise to a doubt as to how far the present conclusions, as to the capacity and life of filter beds, may be depended upon.

RUDOLPH HERING, M. Am. Soc. C. E.—Mr. President, I would like to state, in answer to the gentleman who asked a few questions, that on page 355, the paragraph beginning the 8th line from the bottom gives the answer to one of them.

"With the material best adapted to the purification of large quantities of sewage there is a deposit of nitrogenous matter in the upper layers amounting to about 15% of all of this kind of matter applied in the sewage. The other 85% is oxidized and passes off in solution as nitrates in the effluent. The effect of this deposit on and near the surface is to decrease the air space between the grains of sand and increase the amount of water continually held by these layers, thus in time preventing sufficient change of air in the layers beneath. Continued efficient action may be had by frequently raking over the surface to the

depth of 1 in.; but there comes a time when the upper inch should be removed and replaced by clean sand."

This removal seems to be required under the best conditions. Where the sand is not so perfectly adapted as in this case, the upper layer would have to be more frequently removed.

I would like to add that I consider this paper as being one of the most important that has been prepared on the subject, as it presents, I believe, for the first time to an engineering society, the results gained from these valuable experiments. Formerly, it was supposed that the purification of sewage was accomplished mainly by vegetation, *i. e.*, by plants which were supposed to extract the organic matter from the sewage, and thus purify the water. It has, however, been demonstrated by the experiments of the State Board of Health of Massachusetts that this is not the case, to any great extent, but that we must look for the purification to other means, namely, to pure, clean sand without any organic matter in it. The paper explains why, and the full reports of the Board furnish the proof. I think a marked progress has herewith been made on the subject. We still hear it said, mainly in Europe, that we must *utilize* the sewage in order to properly dispose of it. But that is not the true way to look at the question; we must *purify* it, and when we have done this we may incidentally look to some utilization, but should not consider it the first point to be aimed at. I was in hopes that we might hear from the author also something on the subject of the purification of sewage by dilution, as it is customary in nearly all of our large cities to dispose of the sewage by introducing it into a large body of water. It has been stated in England that sewage will purify itself by flowing a certain distance in a large stream. Some years ago I made a careful investigation of this subject, but failed to find any definite or reliable information. It seemed to me that what was thought to be purification was merely a greater dilution of the sewage by the added quantities of water entering the stream further down. My examinations of several American and European rivers showed that the quantity of albuminoid ammonia contained in the water was almost in proportion to the quantity of pure water that was added to the river. There appeared to be, however, a slight actual purification, probably due to the bacteria that are always present in river water. For all practical purposes this purification seems insignificant; yet, I would like to see some board or some corporation

make the necessary experiments to throw further light on the subject of sewage dilution.

There is another important question which appertains to the general subject, namely, the purification by the addition of chemicals. We have at the World's Fair a plant which receives all the sewage from the grounds, about 2 000 000 galls. a day, I believe. This quantity is received and delivered into several tanks, where it receives certain chemicals, which precipitate the suspended matter, the effluent water running off into the lake. The general system of precipitation was first introduced in England and since then has been introduced in several cities of Europe, and also in our country. The particular appliances used here are of German invention. I was a little surprised to learn of the present result. It seems that occasionally the number of bacteria has been increased, and that the addition of the chemicals does not remove much more than one-half of the organic matter contained in the sewage. Most of the suspended matter is removed, but the organic matter held in solution, which is of course just as objectionable, seems to be left in it.

Mr. HAZEN.—In regard to the treatment of sewage at the World's Fair, I would say that the quantity of sewage is from 2 000 000 to 3 000 000 galls. per day, and under favorable conditions we are removing 60% of the organic matters of the sewage; at other times when the flow is excessively irregular, owing to the action of the Shone ejectors, the percentage is sometimes reduced even below 50 per cent. The removed matters consist of suspended insoluble matters; hardly any of the soluble matters are precipitated. In regard to which of the two forms is most objectionable: We have one problem where a nuisance is caused by sewage forming deposits which decompose and give off offensive odors, as in the Chicago and Milwaukee rivers. We have another problem which is entirely different, and which must be reached in different ways, where sewage is running into drinking water, causing disease. Where it is a question of forming deposits, and producing bad odors; then removing the suspended matters from the sewage will reduce those dangers I think much more than the total reduction of organic matters.

Dr. THOS. M. DROWN.—It is not fair, I think, to say that the statements in the recent reports of the Massachusetts State Board of Health contradict the statements made in the previous reports with regard to

the capacity of soils for the purification of sewage. Each year's work adds to our knowledge of the subject, and gives us information that can only be obtained by additional years' experience with the filters. The experience of two years suggested the probability of the indefinite use of a filter for the purification of sewage, without the renewal of any of the sand. Additional experience shows us that with high rates of application of sewage the accumulation of fine sludge may shut out the air more or less completely from the filter, and retard or prevent oxidation. The remedy in such case may be smaller applications of sewage, a period of rest, or the removal and replacement of the surface layer of sand. On the large scale, with ample areas, which permit the frequent resting of the beds, it would probably never be necessary to remove the surface sand.

With regard to the question whether a sewage disposal area could be made to purify an amount of sewage equal to the amounts indicated in the Lawrence experiments, I would say that they could if they were conducted with equal care and judgment. But this is not to be expected on the large scale. In the forthcoming report of the Board this point receives due consideration, and it is said that in regular practice we cannot expect the process of sewage purification to be carried on with the scientific supervision which is given to the experimental filters at Lawrence.

It must be borne in mind that the Lawrence work is purely experimental. We are studying the problem on a large scale by laboratory methods, and with the highest scientific precision, in order that we may learn the nature of the process and determine its applicability under varying conditions to the purification of sewage on a large scale.

G. H. Benzenberg, M. Am. Soc. C. E.—It seems to me that the experiments which have been made at Lawrence by the State Board of Health do determine that sewage can be purified by the processes of intermittent filtration. If a plant is designed with that view of disposing of the sewage, the engineer must be careful not to admit the sewage too fast upon the filtration beds, thereby clogging the action of Nature, but must make allowance for a sufficient surface of filtering beds to permit Nature to do its work in an easy way, without clogging or obstructing. At Lawrence, undoubtedly, the experimental corps have been able to obtain larger results than would be obtained under ordinary conditions. It seems to me it would not be wise to follow

the proportions that have been used at Lawrence, but to allow a larger filtration area of sand when considering the question of the disposal of sewage by a filtering plant. Those experiments seem to demonstrate that the work can be done in that manner, as it will be done everywhere by the forces of Nature, if the danger of clogging or overcharging the areas is not allowed.

Robert Moore, M. Am. Soc. C. E.—I cannot refrain from adding my testimony of appreciation to the extraordinary value of the experiments, the results of which are recorded in the paper by Mr. Mills. I think there is no question but that the experiments here recorded, made under the auspices of the Massachusetts State Board of Health, exceed in value all that have been made before anywhere. They have advanced our knowledge from uncertainty and doubt to a condition of positive certainty. The value of these researches is increasing every day. A large number of towns in our country are finding it necessary to take steps to purify, not only their sewage, but their water supply, and the knowledge gained by these researches of the different methods in which this can be done places the whole country in debt. I cannot, moreover, help feeling a sense of regret that these researches had not been made somewhat earlier, or if not, that the steps which the city of Chicago is now taking to further pollute the Illinois River had not been for a few years deferred. To those of us who live on this river, or below its mouth, this work which Chicago is now doing causes much solicitude. The pollution of this river, which is due in part to the towns along its banks, but primarily to the sewage already poured into it by Chicago, is yearly becoming greater, and, in low water, is distinctly traceable in the Mississippi below the mouth of the Illinois. In fact, we begin to suspect that St. Louis is indebted to this cause for a large part of its typhoid fever. But be this as it may, to break through a natural divide in order to pollute the water of a foreign stream, as Chicago is now doing, and preparing to do in much greater degree, is a step which can be justified only by overwhelming and absolute necessity. The facts set forth by Mr. Mills, however, afford a complete demonstration that, by filtering its sewage or its drinking supply, or both, Chicago could have protected itself without resorting to the unusual, not to say unnatural, means to which it has in fact resorted. I repeat, therefore, my regret that this demonstration had not come a little earlier, or the action of Chicago been a little longer delayed.

Mr. HERING.—I might answer this remark by stating that even if the results had been known it would not have made any difference in the plan for disposing of the Chicago sewage; enough was known to reach a safe conclusion in that case. England has purified sewage for 30 or 40 years by filtration, and has given us a great deal of practical experience as to what could be done by it. Mr. Bailey Denton has advocated intermittent filtration since the year 1875, or perhaps earlier, and based the same on results obtained by Dr. Frankland. I think it is due to Mr. Denton to state that the practical success of intermittent filtration was first accomplished by him. We knew all this at the time the investigation was made here in Chicago, and I might add that the estimated cost of purifying the sewage by this means was found to require the expenditure of something like $60 000 000. There is no sand in the immediate neighborhood of Chicago that is fit for the purpose. It would have been necessary to build large sewers to the southern end of Lake Michigan and then prepare the sand dunes which are found there, and convert them into filter beds. The estimated cost was about $60 000 000, and that put the adoption entirely out of question.

Mr. E. C. JORDAN.—I think we owe a great debt to Massachusetts for lifting us out of the condition that we were put into as to the utilization of sand by a former National Board of Health. I recollect the papers that were written at that time, stating quite graphically the way in which the sand was used or experimented with, that it offered no resistance whatever to the passage of bad germs, and that we were not at liberty to consider it a method or means of filtration. We have, however, gone back to the old rules of Nature that we have seen going on in every little stream and water supply. We have wondered if the results which were given to the public in that report were true. Now comes the Massachusetts Board of Health and tells us that we were originally right and they have established a formula for it. At the most, it is simply the formula of what we have always observed. I feel quite gratified at being able to go back to the original ideas that we have been educated in by Nature, and we now can reasonably expect good work from the results indicated and approved by the Massachusetts Board.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

685.

(Vol. XXX.—December, 1893.)

THE SEWERAGE SYSTEM OF MILWAUKEE.—DISCUSSION ON PAPER No. 651.*

By D. J. WHITTEMORE, F. P. STEARNS and G. H. BENZENBERG.

D. J. WHITTEMORE, Past. Pres. Am. Soc. C. E.—I wish, as coming from Milwaukee, to make some remarks upon the paper by Mr. Benzenberg. I wish to call the attention of this Congress to the condition of the Milwaukee River before the work of flushing commenced, and which the writer of this paper, through his natural modesty, has not adequately emphasized. Along several miles of the river many of the best and most extensive mercantile and other establishments are located, covering the ground between the shore and adjacent streets completely. Into the river was flowing 60% of the sewers of a city of 200 000 population, and of course the river was foul. This is a mild term. It was so filthy from noxious and nauseating exhalations that I venture to say Chicago River would be termed clean in comparison. A person could not pass over it without having a feeling of nausea. Occupants of the buildings were made ill. Removals from its banks were contemplated by many, and real estate in its proximity depreciated in value.

* "The Sewerage System of Milwaukee, and the Milwaukee River Flushing Works" By G. H. Benzenberg, M. Am. Soc. C. E. *Transactions*, Vol. XXX, page 367.

While the people universally demanded a remedy, a majority had little faith in the plan proposed by the writer of the paper under consideration. Having had some experience with centrifugal pumps, and knowing the general dimensions of the work proposed, I myself had grave doubts of its success. I knew the duty developed by the Bridgeport pumps in this city, and pumps of like nature used to drain marshes in Italy; hence I came to the conclusion that with no greater duty and efficiency than those, the project proposed by the writer of the paper under consideration would prove a failure. What was the result? Within one day after starting the work the nuisance was completely abated, through the agency of pumps and machinery which developed a duty of about 76 000 000 ft.-lbs. per 100 lbs. of coal used, being quite double the duty developed by any pumps of somewhat similar nature of which I then had or now have knowledge. Informing a distinguished German engineer who had made machines of this nature a subject of careful inquiry (Edward Hamberle, of Carlsruhe) of the result of this work, he replied that the people of Milwaukee had the most efficient centrifugal pump in the world. It is not often that the engineer's work is so heartily appreciated within 24 hours after its completion as was that of the writer of the paper under consideration, by the people of Milwaukee.

In this connection it may be proper to ask why the city of Chicago, whose authorities naturally must have known of the existence of this pump and of its remarkable efficiency, has recently attempted to employ an untried device for a similar purpose, which attempt, as I am reliably informed, has resulted in lamentable failure.

F. P. Stearns, M. Am. Soc. C. E.—I wish to express my appreciation of the value of Mr. Benzenberg's paper. It is a very complete and concise description of a very interesting system of works. I have been particularly interested in the centrifugal and screw pumping machinery at Milwaukee, both on account of the large capacity of the pumps and the high duty obtained with them. The duty obtained with the centrifugal pump is, so far as I know, much higher than was ever before obtained with a pump of this character, and the results have raised this class of machinery very much in my estimation, for sewerage works, where as a rule only a low lift is required. In large works there is a very great saving in the first cost of foundations and pump wells as well as machinery by using centrifugal instead of reciprocating pumps.

In making preliminary estimates of the cost of the new metropolitan system of sewerage for the cities and towns in the vicinity of Boston, it was found that it would be very much cheaper to put in two pumping stations on the line of the main sewer, lifting the sewage one-half of the total height at each, with centrifugal machinery, rather than to pump to the full height with reciprocating engines at a single pumping station. The use of two pumping stations in this case was particularly advantageous, because it permitted the sewer (about five miles long) between them, to be laid at a moderate depth below the surface of the ground instead of at a very great depth. It is worthy of note, however, that the estimated cost of the two pumping stations with centrifugal machinery was much less than the cost of a single station with twice the lift, provided with reciprocating pumps.

G. H. Benzenberg, M. Am. Soc. C. E.—In connection with the expense of forcing the water through the flushing tunnel, that question having been asked, I found it was not very much more than that of a high duty water-works pumping engine, that is, about 3¾ cents or 3.78 cents per 1 000 000 galls. per foot. That is, upon a basis of using the best quality of hard coal, which cost, say, $5 15 to $5 25 per ton, delivered at the station. That quality of coal is used because the pumping station is situated in one of the finest residence sections of the city; where, if soft coal were used, the people would object. It makes the cost of pumping of course much higher than if a cheaper grade of coal were used.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

686.

(Vol. XXX.—December, 1893.)

THE NORTH SEA CANAL OF HOLLAND.—DISCUSSION ON PAPER No. 652.*

By J. FÜLSCHER, R. W. LESLEY, THEODOR REHBOCK and W. R. HUTTON.

Mr. GLEIM (translating remarks by Mr. J. Fülscher).—Mr. Fülscher would like to ask a few questions. On page 399 (Vol. XXX), the author says that on completing the foundations of the locks the layer of concrete, which was 2.5 m. high, had been covered with 2 m. of sand. Mr. Fülscher would like to ask why this was thought necessary or advisable, and whether it would not have been as well to allow the concrete to harden under water, without covering it with the sand.

In the second place, Mr. Fülscher refers to the fact that for the mixing of the concrete the author specified two-thirds of the broken brick material to be hard-burned, or else rock, granite or other hard natural stone, and says that he does not see the reason for this, and asks whether it might not have been as well to use ordinary brick. Considerations of specific gravity have apparently not been the cause, or else the broken bricks would not have been admitted at all. It must therefore be inferred that the specifications were induced by considera-

* "The Enlargement and Improvement of the North Sea Canal of Holland (Amsterdam Ship Canal)." By A. E. Kempees, C. E., Royal Engineer in Holland. *Transactions*, Vol. XXX, page 386.

tions of greater crushing strength; but it is not apparent why this case should have required a more than ordinary amount of crushing strength.

R. W. Lesley, Assoc. Am. Soc. C. E.—In reply to Mr. Fülscher as to this point, I would state that possibly the reason for using hard-burned brick clinker rather than the ordinary light-burned brick in making concrete was the fact that where concrete is made of light-burned brick the brick absorb the water very rapidly from the concrete, ultimately causing the disintegration of this latter.

In Philadelphia there was recently a case of this kind where a large amount of concrete was made of brickbats, cement and sand. This was done in hot weather, and the brick absorbed the water from the cement mortar so rapidly that in a short time after the work was done it all began to fall away, and the piers had to be rebuilt. For this reason, no doubt, the specification was made requiring the use of hard-burned brick clinkers rather than soft-burned brick.

Mr. Gleim.*—Mr. Fülscher states that if the hard material had been specified from a fear of the brick absorbing too much water from the mortar in mixing the concrete, he does not agree with this view, especially in a case where the concrete was deposited under water.

For a third question, Mr. Fülscher refers to the fact mentioned on page 400, that springs were found in the soil when the concrete was being put in, and that the places of these springs were just where the bore holes to test the soil had been made. He asks if that could not have been avoided by making the bore holes around the site of the lock instead of inside. As the author is not here, that is a question which could hardly be answered by an outsider.

Theodor Rehbock replied in German (translated by Mr. Gleim) that he happened to know from one of the engineers of the work that the bore holes were not made for this canal, but had been made for other work before this line of canal had been settled; it was a mere accident that this lock happened to come in this place.

Mr. Gleim.*—Mr. Fülscher says he has had a similar experience in his work, which is the great North and East Sea Ship Canal in Germany, at places where the bore holes had been made at a time when the situation of the lock was not finally fixed. The speaker quotes the means they took in treating these springs. The paper says they had allowed

* Translating Mr. Fülscher's remarks.

the water to rise to a moderate amount only before putting in the concrete. He asks whether they could not have allowed it to rise to a greater height, so as to balance the pressure of the springs and have no movement of the water at all. By this means they might possibly have avoided the troubles which they afterwards had on this account, because the concrete was washed out by the springs.

Then, after those springs had given trouble, the damaged concrete was taken out at those places, and large standpipes put in, with flanges underneath, which were embedded from the outside in concrete. Mr. Fulscher thinks it would have been better to have taken this measure from the beginning and inserted those pipes before putting in the concrete as they have always done in similar cases on his work.

He further discusses the plan of closing up those pipes with concrete as the work proceeded, and asks why this was thought necessary, and whether it would not have been better to have left them open, because in case the lock is ever pumped out for the sake of repairs, he thinks those closed-up pipes may be dangerous, unless the floor of the lock be considered strong enough to withstand the pressure of those sealed-up springs against the concrete bottom.

W. R. Hutton, M. Am. Soc. C. E.—There are numerous points of interest connected with the North Sea Canal and its harbor works.

First may be noted the great efficiency of a wide, thin platform of rip-rap under the pier walls. The work has commenced, as stated by Mr. Kempees, by laying the concrete blocks directly upon the sand, where they were promptly undermined, and the entire work accomplished was threatened with destruction.

After this a platform of small rubble (rip-rap) was established about 100 ft. in width and 3½ ft. thick. The pier walls built upon it with a width at base of 20 to 27 ft. have never been disturbed, and Mr. Darnton Hutton (engineer of the contractors who constructed the works) informs me that no displacement of the rip-rap has ever been observed. It is true that the outer half of the length of the piers which is exposed to the heavier seas is protected on the seaside by "wave breakers," concrete blocks weighing 20 tons, disposed as a rip-rap.

This foundation has since been applied in other harbor works; rarely, however, it is believed, on our own coasts. The late Mr. Henry Bacon, civil engineer for many years in local charge of the Cape Fear

River improvement, has expressed the opinion that it is the best bottom protection, even on those shifting sands.

Manora and Columbo breakwaters have been erroneously cited as instances of the above construction. Those were mounds of rubble, built up to within a few feet of the water-surface and surmounted by a masonry wall. The basalt bed at Ymuiden is purely a foundation work; its purpose is to protect the base of the wall and prevent undermining by scour. In 1878 it was observed that the earlier concrete blocks were much worn by the action of the sea. Those made later with a larger proportion of cement, although they had been exposed for two years to the action of very heavy waves, still retained the sharpness of their angles. From Mr. Kempees' paper it is inferred that these, too, have suffered with lapse of time.

A second point of interest is the large quantity of sand brought into the harbor by the tide. The original plan of the harbor provided an elliptical basin occupying nearly all the space inside the piers, in which ships could lie at anchor. It is probable that much of the dredging of the earlier years after completion was done in the effort to make this basin. In 1878 it was said at the works that, although the flood current split on the North Pier Head, but little sand was brought in. The late Mr. Dirks, chief engineer of the works in 1883, spoke most discouragingly of the large deposits as a somewhat recent discovery.

The flood tide sweeps along the coast of Holland, northeasterly, with a velocity of 2¼ to 2½ miles an hour—about 4 ft. a second; the ebb current is much weaker, and is not parallel with the coast line. At the Hook of Holland the bottom currents vary much in direction from those of the surface for some time before and after either turn of the tide. The heaviest storms are from the northwest.

The deposit is apparently not caused by the movement of the sands along the beach as was predicted by Croizette de Noyers in 1873, for the angle of the pier with the beach is not filled up, the depth curves are not pushed farther out, and there is a scour along both the canted or oblique portions of the harbor walls. The large deposits inside the harbor would seem to come from sand carried in suspension by the strong flood current and dropped in the quiet water inside the pier.* Their principal location along the inside of the north pier is further

* See, also, Mr. Harrison Hayter in Proceedings, Inst. C. E., Vol. LXII.

evidence that the flood current splits on the north pier and enters with considerable velocity.

Sir John Hawkshaw who designed the harbor, at a later period maintained that an artificial harbor in the sea in strong littoral currents, as in the English Channel and at this place, should have a long front parallel with the shore line, with the entrance in the middle of its length, as then proposed for the harbor of Dover and since applied in the new harbor of Boulogne. He is quoted as having said that the Ymuiden harbor was not designed on this plan because there were not sufficient funds at his disposition.

No experiments are known to have been made to ascertain the amount of sand carried by these currents, or others in similar conditions. Roughly computed from the dredgings it would appear to be here about $\frac{1}{700}$ of their volume. The Mississippi River in flood carries silt to about $\frac{1}{1300}$ of its volume. In the estuary of the Seine, currents of 10 ft. per second, rushing over shallow banks of silt, take up at times $\frac{1}{150}$ of their volume of light material, although the average for a flood tide is something less than $\frac{1}{1800}$ by volume.

The following suggestion is made with diffidence, for want of local knowledge, observation and experience:

The general maximum velocity of the tidal water in the entrance, due only to the rising of the tide, would be 0.3 ft. per second. If a portion of the flood tide current enters with a velocity of 3 or 4 ft. per second, some of it must flow out again through the entrance, either at the bottom or around the south pier head, for the water in the harbor cannot be sensibly higher than that outside. If this be so, the deposits come from the material carried in suspension by a much larger quantity of water than would fill the harbor to high-water line. If the flood current can be prevented from entering with velocity beyond that actually necessary to fill the basin, the quantity of deposit will be largely reduced.

The scour along the canted arms of the harbor walls seems to indicate that the mass of water nearer the shore, obstructed by the piers, flows with greater velocity along the south cant, crosses the entrance, a portion being deflected into the harbor, and follows along the north cant into the otherwise quiet water north of the harbor. This would probably have been different if the wall north of the entrance had been continued parallel with the sea current, even for a short

distance. If all this be true, it does not necessarily follow that it should be corrected now. The cure may be more expensive than the disease.

The project of Mr. Huet for an open canal from the North Sea to the Zuider Zee is not sufficiently described in his paper.

It has elicited much discussion in the Netherlands Society of Engineers, and a project has been prepared which it would be of great interest to discuss. Some lively currents would probably be inaugurated, owing to differences of tidal levels in the two seas, and it would be of interest to consider their effect upon the silting of the harbor.

The bar, off the entrance, mentioned by M. Quinette de Rochemont, may be composed of the material scoured from the base of the south cant, or it may be formed by wave action normal to the shore line.

LIST OF PAPERS PRESENTED TO DIVISION A, INTERNATIONAL ENGINEERING CONGRESS, IN CHARGE OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

PAPERS PRESENTED AND PUBLISHED.

"Navigation Works Executed in France from 1876 to 1891."
By F. Guillain, Inspecteur-Général des Ponts et Chaussées, Paris, France, xxix, p. 1.

"Inland Transportation."
By Capt. F. A. Mahan, Corps of Engineers, U. S. A., xxix, 97.

"History of the Conversion of the River Clyde into a Navigable Water-Way, and of the Progress of Glasgow Harbor from its Commencement to the Present Day."
By James Deas, Engineer Clyde Navigation, xxix, 128.

"Description of the Lower Weser and its Improvement."
By L. Franzius, Ober-Bau-Director, Bremen, Germany, xxix, 173.

"A Brief Account of the Building of Leixões Harbor."
By Alfonso Joaquim Nogueira Soares, Portugal, Inspecting Engineer in Charge, xxix, 194.

"The Limits Attainable in Improving the Navigability of Rivers by Means of Regulation."
By H. Engels, Professor of Hydraulics in the Royal Technical High School, Dresden, Germany, xxix, 202.

"The Improvement of Harbors on the South Atlantic Coast of the United States."
By William Murray Black, Captain Corps of Engineers, U. S. A., xxix, 223.

"The Rearrangement of Railroad Tracks and Stations in Cologne, Prussia."
By F. Lohse, Eisenbahn Bau und Betriebs-Inspector, Cologne, Prussia, xxix, 277.

"The Rearrangement of the Railway Terminal System at Altona, with Special Reference to the Avoidance of Grade Crossings."
By R. Caesar, Regierungs und Baurath, Member of the Royal Board of Railroad Directors, Altona, Prussia, xxix, 295.

"Common Roads, Railways and River Communications in Portugal."
By Frederico Augusto Pimental, Civil Engineer, Lisbon, Portugal, xxix, 299.

"The Railway System of New South Wales."
By Thomas Fletcher Birrell, Civil Engineer, Newcastle, New South Wales, xxix, 326.

"Railroads in the Republic of Mexico in 1893."
By E. Prieto Basave, M. Association of Engineers and Architects of Mexico, xxix, 357.

"Study Upon Railroads to Connect Guadalajara with the Pacific Ocean."
By S. V. Pascal, M. Association of Engineers and Architects of Mexico, xxix, 373.

"Distinctive Features and Advantages of American Locomotive Practice."
By David L. Barnes, M. Am. Soc. C. E., xxix, 385.

"Comparison of Modern Engine-Loading with Standard Specifications for Spans from 10 to 200 Ft."
By C. D. Purdon, M. Am. Soc. C. E., xxix, 426.

"Surveys for Railway Location."
By F. A. Gelbcke, Engineer of Government Railways, Cologne, Prussia, xxix, 429.

"A New Method of Calculating Cross-Sections of Roads and Railroads."
By Francisco Da Silva Ribeiro, Director of Public Works, Aveiro, Portugal, xxix, 447.

"On the Gauges of Railroad Track in General, with Special Consideration of Narrow-Gauge Railroads."
By E. A. Ziffer, C. E., Vienna, Austria, xxix, 453.

"Railway Signaling."
By G. Kecker, Betriebs-Director Imperial Railways, Metz, Germany, xxix, 491.

"Transmission of Power in Operating Cable Railways."
By Robert Gillham, M. Am. Soc. C. E., xxix, 543.

"The Continuous Superstructure of the Memphis Bridge."
By George S. Morison, M. Am. Soc. C. E., xxix, 573.

"Bridge Substructure and Foundations in Nova Scotia."
By Martin Murphy, M. Can. Soc. C. E.; President N. S. Inst. Sc.; Provincial Government Engineer, Nova Scotia, xxix, 620.

"Improved Method of Constructing Foundations Under Water by Forcing Cement into Loose Sand or Gravel by Means of Air Pressure."
By Fr. Neukirch, C. E.. Bremen, Germany, xxix, 639.

"American Grain Elevators."
By G. Lee Heidenreich, M. Scandinavian Engineering Society of Chicago; Am. Inst. Min. Engrs.; Western Society of Engineers, xxix, 644.

"The Manufacture and Use of Paving Brick."
By Daniel W. Mead, M. Western Society of Engineers; Assoc. M. Am. Soc. C. E., xxix, 653.

"Carbon and its Uses in Electrical Engineering."
By Clarence M. Barber, C. E., M. Civil Engineers' Club of Cleveland, xxix, 680.

"The Guadalajara Electric Light Installation, Utilizing the Famous Juanacatlan Water-Falls, 28 Km. Distant from Guadalajara."
By Rafael M. de Arozarena, Mechanical Engineer, M. Am. Soc. Mech. Engrs.; M. Association of Engineers and Architects of Mexico, xxix, 689.

"The Relative Merits of Working Hoisting Machinery by Steam, Water or Electricity."
By George A. Goodwin, Senior Vice-President of the Society of Engineers; Assoc. M. Inst. Civil Engineers; Whitworth Scholar, xxix, 695.

"Practical and Æsthetic Principles for the Laying Out of Cities."
By J. Stübben, Baurath and Assistant Burgomaster, Cologne, Germany, xxix, 718.

"The Testing of Portland Cement and the Development of the Cement Industry in Germany."
By Max Gary, C. E., Berlin, Germany, xxx, 1.

"On the Manufacture and Testing of Portland Cement."
By Henry Faija, M. Inst. C. E., xxx, 43.

"Topographic Surveys."
By Herbert G. Ogden, Assistant U. S. Coast and Geodetic Survey, xxx, 62.

"Recent Experience on the U. S. Coast and Geodetic Survey in the Use of Long Steel Tapes for Measuring Base Lines."
By R. S. Woodward, C. E., xxx, 81.

"Terrestrial Magnetism in North America."
By Charles A. Schott, Assistant U. S. Coast and Geodetic Survey, xxx, 108.

"Fundamental Units of Measure."
By T. C. Mendenhall, Superintendent of the U. S. Coast and Geodetic Survey, xxx, 120.

"Historical Notes upon Ancient and Modern Surveying and Surveying Instruments."
By H. D. Hoskold, M. and C. E., Inspector-General of Mines, Argentine Republic, xxx, 135.

"The Treatment of Metals for Structural Purposes."
By James Christie, M. Am. Soc. C. E., xxx, 155.

"A Proposed Method of Testing Structural Steel."
By Alfred E. Hunt, M. Am. Soc. C. E., xxx, 181.

"The Use of Mild Steel for Engineering Structures."
By George C. Mehrtens, Regierungs und Baurath, Bromberg, Prussia, xxx, 204.

"The Use of Basic Mild Steel as Material for Construction in Germany."
By C. Weyrich, Engineer of River and Harbor Board, Hamburg, Germany, xxx, 219.

"The Plant of Maritime Commercial Ports of France."
By H. Despres, Ingenieur des Ponts at Chaussées, xxx, 235.

"The Development of Quay-Cranes in the Port of Hamburg."
By Chr. Nehls, Chief Engineer of River and Harbor Board, Hamburg, Germany, xxx, 258.

"Method Used to Secure the Stability of a Quay Wall at the Port of Altona, on the Elbe, in Germany, which had Shifted its Position After Completion."
By Berthold Stahl, Stadtbaurath, Altona, Germany, xxx, 284.

"Experiences Had During the Last Twenty-Five Years with Water Works Having an Underground Source of Supply."
By B. Salbach, Baurath, Dresden, Germany, xxx, 293.

"Some Questions Concerning the Filtration of Water."
By W. Kümmel, Engineer of Gas and Water Works Company, Altona, Germany, xxx, 330.

"On the Distribution of Water in the City of Mexico."
By L. Salazar, M. Association of Engineers and Architects of Mexico, xxx, 336.

"Purification of Sewage and of Water by Filtration."
By Hiram F. Mills, A. M., C. E., xxx, 350.

"The Sewerage System of Milwaukee and the Milwaukee River Flushing Works."
By G. H. Benzenberg, M. Am. Soc. C. E., xxx, 367.

"The Enlargement and Improvement of the North Sea Canal of Holland (Amsterdam Ship Canal)."
By A. G. Kempees, C. E., Royal Engineer, Holland, xxx, 386.

"Notes on Projects for the North Sea Canal from 1629–1893."
By A. Huet, C. E., xxx, 416.

"The North and East Sea Canal."
By J. Fülscher, Member of Imperial Board of Canal Commissioners, Kiel, Germany, xxx, 421.

"The New Railway Stations at Dresden."
By C. Köpcke, Geheimer Finanzrath, and O. Klette, Baurath, Dresden, Germany, xxx, 450.

PAPERS PRESENTED BY TITLE.

"On the Possibility of Increasing the Water Supply from the Soil for Purposes of Irrigation and of Augmenting the Flow of Rivers."
By G. H. Otto Volger, Ph.D., M. German Association of Civil Engineers' Societies.

"The Calculation of Railroad Bridge Stresses."
By John L. P. O'Hanly, M. Am. Soc. C. E.

"The Tequixquiac Tunnel."
By Louis Espinoza, M. Association Architects and Engineers of Mexico.

"The Sewerage and Drainage of the Town of Coimbra (Portugal)."
By J. C. DaCosta, J. DaC. Couraça and J. F. de M. Bessa, Engineers, Members Society of Civil Engineers of Portugal.

"The Improvement of the Harbor of Lisbon."
By B. J. J. de Mattos, M. Society of Civil Engineers of Portugal.

"A Method of Making Mountain Railways Economical."
By Ugo Baldini, M. Society of Italian Engineers and Architects, Rome, Italy.

"Determination of Uniform Methods for Testing Building and Construction Materials."
By Prof. J. Bauschinger, Munich, Germany.

"The Best Form of Level Book."
By A. T. Walmisley, M. Inst. C. E.

"On Rigid Arch and Suspension Bridges."
By C. Köpcke, Geheimer Finanzrath, Dresden, Germany.

"On the Desirability of a General Adoption by all Countries of a Uniform Method for the Tests of Materials of Construction."
By M. Debray, Chief Engineer Ponts et Chaussées, Paris, France.

"Data and Observations of the Work under Construction for the Improvement of the Harbor of Vera Cruz."
By Miguel A. de Quevedo, M. Association of Engineers and Architects of Mexico.

"The Channel Bridge."
By L. Violette, Fellow of the Society of Civil Engineers of France.

"Les Ports de Mer dans l'Archipel des Asores."
By T. V. Mendes Guerreiro, M. Society of Civil Engineers of Portugal.

"National Railroad of Tehuantepec."
By Manuel Rivera, M. Association of Engineers and Architects of Mexico.

"The Sanitary Problem of Chicago."
By L. E. Cooley, C. E.

TRANSACTIONS

OF THE

American Society of Civil Engineers.

INDEX.

VOLUMES XXIX AND XXX.

JULY TO DECEMBER, 1893.

NOTE.—In this Index, Subjects and Writers are given Alphabetically. Titles of Papers are enclosed by quotation marks when indexed alone or under the Author's name. When indexed under the name of a person discussing the paper, the titles are not so enclosed. Titles of papers are entered with reference to the subject and also under the first significant word.

INDEX.

VOLUMES XXIX AND XXX.

JULY TO DECEMBER, 1893.

ALLEN, KENNETH.
Topographic Surveys, xxx, 612.
Translation of paper by H. Engels, xxix, 202.
Translation of paper by J. Fülscher, xxx, 421.

ALTONA, GERMANY.
Quay Wall at— Berthold Stahl, xxx, 284.
"Railway Terminal System at—" R. Caesar, xxix, 295; C. O. Gleim, xxx, 512.

ALUM.
Use of—for Purifying Water. B. Salbach, xxx, 298.

"AMERICAN GRAIN ELEVATORS." E. Lee Heidenreich, xxix, 644.
Discussion, xxx: Edw. B. Taylor, 584; E. Lee Heidenreich, 584.

AMSTERDAM SHIP CANAL.
A. E. Kempees, xxx, 386; A. Huet, xxx, 416.

ARCHIBALD, P. S. Bridge Substructure and Foundations in Nova Scotia, xxx, 574.

AROZARENA, RAFAEL M. DE. "The Guadalajara Electric Light Installation, utilizing the famous Juanacatlan Water Falls, 28 km. distant from Guadalajara," xxix, 689.

ARTESIAN WELLS.
In Mexico. L. Salazar, xxx, 341.
In Silesia. B. Salbach, xxx, 309.

BACTERIA.
Destruction of— H. F. Mills, xxx, 358.

BAKER, Sir BENJAMIN.
Continuous Superstructure of the Memphis Bridge, xxx, 561.
Manufacture and Testing of Portland Cement, xxx, 603.

BAKER, MARCUS. Topographic Surveys, xxx, 619.

BALDWIN, WARD. Loadings for Railway Bridges, xxx, 515.

BARNES, David L. "Distinctive Features and Advantages of American Locomotive Practice," xxix, 385.

BARBER, Clarence M. "Carbon and Its Uses in Electrical Engineering," xxix, 680.

BASAVE, E. Prieto. "Railroads in the Republic of Mexico in 1893," xxix, 357.

BASE LINES.
Measurements in England. H. D. Hoskold, xxx, 141.
Measurements in France. H. D. Hoskold, xxx, 139.
Measurements in Peru. H. D. Hoskold, xxx, 139.
Use of Long Steel Tapes in Measuring— R. S. Woodward, xxx, 81.

BEAHAN, Willard. Manufacture and Use of Paving Brick, xxx, 586.

BENZENBERG, G. H.
Purification of Sewage and of Water by Filtration, xxx, 706.
"The Sewerage System of Milwaukee and the Milwaukee River Flushing Works," xxx, 367, 711.

BEYER, Albert. Translation of paper by L. Franzius, xxix, 173.

BIRRELL, Thomas Fletcher. "The Railway System of New South Wales," xxix, 326.

BLACK, Wm. Murray.
"Improvement of Harbors on the South Atlantic Coast of the United States," xxix, 223; xxx, 500, 509.
Improvement of Rivers, xxx, 486, 492.
Manufacture and Testing of Portland Cement, xxx, 601.

BONTECOU, D. Transmission of Power in Operating Cable Railways, xxx, 550.

BREAKWATERS.
Holyhead. John P. Griffith, xxx, 489.
In Leixões Harbor. A. J. N. Soares, xxix, 195.
North Sea Canal of Holland. A. E. Kempees, xxx, 393.

BRICK.
Analysis of Different Clay for— D. W. Mead, xxix, 659.
Character of Clay for— D. W. Mead, xxix, 656.
Distribution of Clay for— D. W. Mead, xxix, 654.
Durability of— D. W. Mead, xxix, 677.
Qualities of— D. W. Mead, xxix, 665.
Resisting Strength of— D. W. Mead, xxix, 675.
Specifications for— D. W. Mead, xxix, 671.
Testing of— D. W. Mead, xxix, 668.
"The Manufacture and Use of Paving Brick." D. W. Mead, xxix, 653. Discussion, xxx, 585.

BRIDGES.
Avon Bridge, N. S., M. Murphy, xxix, 623, 628; K. Sutherland, xxx, 568; W. R. Butler, xxx, 571.
Bridgewater Bridge, N. S., M. Murphy, xxix, 630.
"Bridge Substructure and Foundations in Nova Scotia." Martin Murphy, xxix, 620.
Discussion, xxx: K. Sutherland, 567; F. W. W. Doane, 569; W. R. Butler, 571; E. Gilpin, 573; P. S. Archibald, 574; H. S. Poole, 576; Martin Murphy, 576.
Concrete used in the Construction of— M. Murphy, xxix, 621.
First Use of Cast Steel for— Geo. C. Mehrtens, xxx, 207.
Gaspereaux Bridge, N. S. K. Sutherland, xxx, 568.
Loadings for Railway— Ward Baldwin, xxx, 515; J. A. L. Waddell, 522; C. D. Purdon, 523.
Mild Steel— Built in Europe, 1880–90. Geo. C. Mehrtens, xxx, 218.
Sutherlands River Bridge, N. S. M. Murphy, xxix, 634.
"The Continuous Superstructure of the Memphis Bridge." Geo. S. Morison, xxix, 573; xxx, 564.
Vibration in— Geo. H. Thomson, xxx, 560; Benj. Baker, 561; Geo. S. Morison, 564.
Victoria Bridge, N. S. M. Murphy, xxix, 632; F. W. W. Doane, xxx, 570.

"BRIEF ACCOUNT OF THE BUILDING OF LEIXÕES HARBOR." Alphonso Joaquim Nogueira Soares, xxix.
Discussion, xxx: John Purser Griffith, 490.

BRUSH, Chas. B.
The Use of Long Steel Tapes, xxx, 639.
Topographic Surveys, xxx, 611.
Underground Sources of Water Supply, xxx, 690.

BRYSON, A.
Operating Cable Railways, xxx, 548.
Surveys for Railway Location, xxx, 528.

BUCK, L. L.
Proposed Method of Testing Structural Steel, xxx, 667.
Treatment of Metals for Structural Purposes, xxx, 655.

BUTLER, W. R. Bridge Substructure and Foundations in Nova Scotia, xxx, 571.

CAESAR, R. "Rearrangement of the Railway Terminal System at Altona, with Special Reference to the Avoidance of Grade Crossings," xxix, 295.

CAISSONS.
In Glasgow Harbor. James Deas, xxix, 153.

CALKINS, F. A. The Manufacture and Use of Paving Brick, xxx, 586.

CANALS.
Amsterdam Ship Canal. A. E. Kempees, xxx, 386; A. Huet, 416.
Decadence of— F. A. Mahan, xxix, 105.
Difference between—and Railroads, F. A. Mahan, xxix, 106.
Erie Canal, Cost of, etc. F. A. Mahan, xxx, 475.
Funicular Towage on— F. Guillain, xxix, 23.
Forth and Clyde Canal. James Deas, xxix, 131.
"North Sea Canal of Holland, Enlargement and Improvement of—" A. G. Kempees, xxx, 386.
"North Sea Canal. Notes and Projects for—" A. Huet, xxx, 416.
"North and East Sea Canal." J. Fülscher, xxx, 421.
—of France. F. Guillain, xxix, 4.
Tonnage on—of France. F. Guillain, xxix, 28; F. A. Mahan, 118.
Traction on—of France. F. Guillain, xxix, 22.
Transportation, Cost of, on—of France. F. Guillain, xxix, 36.

"CARBON AND ITS USES IN ELECTRICAL ENGINEERING." Clarence M. Barber, xxix, 680.

CEMENT, PORTLAND.
Binding Strength of— Max Gary, xxx, 40.
Blowing of— Henry Faija, xxx, 55.
Composition of— Henry Faija, xxx, 43.
Constancy of Volume of— Max Gary, xxx, 12.
Difference between Hydraulic Lime and— Henry Faija, xxx, 44.
Economic Grinding of— Henry Faija, xxx, 48.
First made in Germany. Max Gary, xxx, 2.
For Foundations under Water. Fr. Neukirch, xxix, 639.
Gauging and Manipulation of— Henry Faija, xxx, 59.
Loss of Weight of—by Abrasion. Max Gary, xxx, 40.
Magnesium, Effect of, in— Max Gary, xxx, 18.
"On the Manufacture and Testing of—" Henry Faija, xxx, 43. Discussion, 594.
Packing of— Max Gary, xxx, 10.
Qualities of— Max Gary, xxx, 9; Henry Faija, 50.
Regulations of German—Manufacturers' Association. Max Gary, xxx, 8.
Requirements of Calculation of Materials for— Henry Faija, xxx, 47.
Sampling of— Henry Faija, xxx, 59.
Sea Water, Effect of, on— Wm. M. Black, xxx, 601.
Setting of— Max Gary, xxx, 10; Henry Faija, 50.
Specific Gravity of— Max Gary, xxx, 37.
Specifications for— Henry Faija, xxx, 60.

"Testing of—and the Development of the Cement Industry in Germany. Max Gary, xxx, 1; Discussion, 594.
Tests of— W. R. Butler, xxx, 572; Martin Murphy, 578; Benj. Baker, 603.

CHRISTIE, JAMES.
Proposed Method of Testing Structural Steel, xxx, 669.
"The Treatment of Metals for Structural Purposes," xxx, 155.

CITIES.
"Practical and Æsthetic Principles for the Laying Out of Cities." J. Stübben, xxix, 718; Discussion: James Owen, xxx, 591.
Hygienic Requisites of— J. Stübben, xxix, 724.

CLAY.
Analysis of Different— D. W. Mead, xxix, 659.
Character of, for Brick. D. W. Mead, xxix, 656.
Mining of— D. W. Mead, xxix, 660.
Occurrence and Distribution of— D. W. Mead, xxix, 654.

COAL.
Analysis of— D. L. Barnes, xxix, 412.

COLOGNE, PRUSSIA.
"Rearrangement of Railroad Tracks and Stations in—" F. Lohse, xxix, 277.
Traffic of Railroad Station of— F. Lohse, xxix, 278.

"COMMON ROADS, RAILWAYS AND RIVER COMMUNICATIONS in Portugal." Frederico Augusto Pimental, xxix, 299.

"COMPARISON OF MODERN ENGINE-LOADING, WITH Standard Specifications for Spans from 10 to 200 Ft." C. D. Purdon, xxix, 426, 523.
Discussion, xxx: Ward Baldwin, 515; J. A. L. Waddell, 522.

CONCRETE.
Use of—in Bridge Substructure. M. Murphy, xxix, 621; Discussion, xxx, 567.

"CONTINUOUS SUPERSTRUCTURE OF THE MEMPHIS Bridge." Geo. S. Morison, xxix, 573.
Discussion, xxx: Geo. H. Thomson, 559; Henry Goldmark, 560; Sir Benjamin Baker, 561; Robert Moore, 562; Chas. Macdonald, 563; Geo. S. Morison, 564; A. D. Ottewell, 566.

CRAIGHILL, WM. P. Improvement of Harbors, xxx, 503.

CRANDALL, C. L. Translation of Paper by F. Guillain, xxix, 1.

CRANES.
Amount of Steam Utilized in Steam— G. A. Goodwin, xxix, 700.
Efficiency of Hydraulic— G. A. Goodwin, xxix, 703.
Efficiency of Traveling—at Hamburg. Chr. Nehls, xxx, 263.

Electric—in England. G. A. Goodwin, xxix, 706; in Hamburg, Chr. Nehls, xxx, 277.
Hand—at Hamburg. Chr. Nehls, xxx, 264, 270.
Quay—at Hamburg. Chr. Nehls, xxx, 258.
Tariffs for Use of—at Ports of France. H. Despres, xxx, 253.
Traveling—at Hamburg. Chr. Nehls, xxx, 260, 264.
Used at Ports of France. H. Despres, xxx, 249.
Used on Glasgow Docks. James Deas, xxix, 146.
Worked by Electricity. G. A. Goodwin, xxix, 705.
Worked by Steam. G. A. Goodwin, xxix, 699.

CROSS-SECTIONS.
"A New Method of Calculating—for Roads and Railroads." Francisco Da Silva Ribeiro, xxix, 447; xxx, 534; Discussion, 533.

CROWELL, Foster.
Gauge of Railway Track, xxx, 538.
Translation of Paper by E. Prieto Basave, xxix, 357.
Translation of Paper by F. Da Silva Ribeiro, xxix, 447.
Translation of Paper by S. V. Pascal, xxix, 373.

CUNNINGHAM, A. C. Proposed Method of Testing Structural Steel, xxx, 669.

CURRENTS.
Action of— Wm. M. Black, xxix, 230, 233.
Tidal— Wm. M. Black, xxix, 233.

DAMS.
Closure—Cape Fear River. W. P. Craighill, xxx, 504.
In Savannah Harbor. Wm. M. Black, xxix, 258.
In Weser River. L. Franzius, xxix, 189.
La Mulatière, France. F. Guillain, xxix, 16.
Poses, France. F. Guillain, xxix, 14.
Suresnes, France. F. Guillain, xxix, 14.
Torcy-Neuf, France, F. Guillain, xxix, 21.

DEAS, James. "History of the Conversion of the River Clyde into a Navigable Water-Way, and of the Progress of Glasgow Harbor from its Commencement to the Present Day, xxix, 128; xxx, 482.

"DESCRIPTION OF THE LOWER WESER AND ITS IMPROVEment." L. Franzius. Translated from the German by T. H. McCann and Albert Beyer, xxix, 173.
Discussion, xxx: Th. Rehbock, 483; C. O. Gleim, 485; Wm. M. Black, 486; W. R. Hutton, 487.

DESPRES, H. "The Plant of Maritime Commercial Ports of France," xxx, 235.

"DEVELOPMENT OF QUAY CRANES IN THE PORT OF HAMburg." Chr. Nehls, xxx, 258.

DIKES.
In Cape Fear River. Wm. M. Black, xxix, 241.
In River Weser. L. Franzius, xxix, 190.
Spur—in Savannah Harbor. Wm. M. Black, xxix, 260.

"DISTINCTIVE FEATURES AND ADVANTAGES OF AMERICAN Locomotive Practice." David L. Barnes, xxix, 385.

DOANE, F. W. W. Bridge Substructure and Foundations in Nova Scotia, xxx, 569.

DOCKS.
Of France. F. Guillain, xxix, 54.
Of Glasgow Harbor. James Deas, xxix, 142, 143, 151.

DREDGING.
In Amsterdam Ship Canal. A. E. Kempees, xxx, 388, 397.
In Cape Fear River. Wm. M. Black, xxix, 240, 244.
In Charleston Harbor. Wm. M. Black, xxix, 249, 251.
In Glasgow Harbor. James Deas, xxix, 134, 140, 148.
In St. John's River. Wm. M. Black, xxix, 263.
In Weser River. L. Franzius, xxix, 187, 192.

DRESDEN. "The New Railway Stations at—" C. Köpcke and O. Klette, xxx, 450.

DRILLING, ROCK.
At Port Colborne, Lake Erie. O. M. Poe, xxx, 479.
In St. Mary's River, Michigan, xxx, 482.
With Water-Jet. O. M. Poe, xxx, 480.

DROWN, Thomas M. Purification of Sewage and of Water by Filtration, xxx, 705.

ELECTRICAL ENGINEERING.
"Carbon and Its Uses in—" C. M. Barber, xxix, 680.

ELECTRIC LIGHTING.
"The Guadalajara Electric Light Installation." R. M. de Arozarena, xxix, 689.

ELECTRICITY.
"The Relative Merits of Working Hoisting Machinery by Steam, Water and Electricity." G. A. Goodwin, xxix, 695.

ELEVATORS.
Electricity Used for Hoisting— G. A. Goodwin, xxix, 704.
Hydraulic Power for— G. A. Goodwin, xxix, 702.

ELEVATORS, GRAIN.
"American Grain Elevators." E. Lee Heidenreich, xxix, 644; xxx, 583.

Cleaning— E. Lee Heidenreich, xxix, 650.
Principal Requirements of— E. Lee Heidenreich, xxix, 652.
Railroad Transfer— E. Lee Heidenreich, xxix, 648.
Storage— E. Lee Heidenreich, xxix, 645.
Terminal— E. Lee Heidenreich, xxix, 648.

ENGELS, H. "The Limits Attainable in Improving the Navigability of Rivers by Means of Regulation," xxix, 202, 492.

"ENLARGEMENT AND IMPROVEMENT OF THE NORTH SEA Canal of Holland (Amsterdam Ship Canal)." A. E. Kempees, xxx, 386.
Discussion, xxx: J. Fülscher, 712; R. W. Lesley, 713; Theodor Rehbock, 713; W. R. Hutton, 714.

EXCAVATION.
Subaqueous Rock— James Deas, xxix, 138; O. M. Poe, xxx, 478, 482.

"EXPERIENCES HAD DURING THE LAST TWENTY-FIVE Years with Water Works Having an Underground Source of Supply." B. Salbach. (Translated from the German by Clemens Herschel.) xxx, 293.
Discussion, xxx: Chas. B. Brush, 690; L. L. Tribus, 694; D. J. Whittemore, 696; D. W. Mead, 696; B. Salbach, 701.

FAIJA, Henry. "On the Manufacture and Testing of Portland Cement," xxx, 43, 607.

FILTRATION OF WATER.
B. Salbach, xxx, 294; W. Kümmel, xxx, 330.
H. F. Mills, xxx, 350.

FLAD, Edward. Purification of Sewage and of Water by Filtration, xxx, 702.

FORMULAS FOR TUNNEL OR CONDUITS. G. H. Benzenberg, xxx, 382.

FOUNDATIONS.
Constructing—Under Water. Fr. Neukirch, xxix, 639; xxx, 579.
Cylinders. M. Murphy, xxix, 630.
For Works in Nova Scotia. M. Murphy, xxix, 620.
General Principles for— M. Murphy, xxix, 627.

FRANCE.
Lighthouses in— F. Guillain, xxix, 59.
Maritime Commerce of— F. Guillain, xxix, 64.
"Navigation Works executed in—from 1876 to 1891." F. Guillain, xxix, 1.
"Plant of Maritime Commercial Ports of—" H. Despres, xxx, 235.
Water-Ways of— F. Guillain, xxix, 4.

FRANZIUS, L. "Description of the Lower Weser and its Improvement," xxix, 173.

FREIGHT RATES.
Comparison of—on Rail and Water-Ways. F. A. Mahan, xxix, 119.
Fall in—on Seven Trunk Lines in the United States. Edw. P. North, xxx, 470.
From Chicago to New York. C. W. Raymond, xxx, 457.
In France. C. W. Raymond, xxx, 458.
In Frankfort, Germany. Edw. P. North, xxx, 468.
On the Great Lakes. Edw. P. North, xxx, 471.
On the Erie Canal. F. A. Mahan, xxx, 475.

FÜLSCHER, J.
"The North and East Sea Canal," xxx, 421.
The North Sea Canal of Holland, xxx, 712.

"FUNDAMENTAL UNITS OF MEASURE." T. C. Mendenhall, xxx, 120.

GANNETT, Henry. Topographic Surveys, xxx, 615.

GARY, Max. "The Testing of Portland Cement and the Development of the Cement Industry in Germany," xxx, 1.

GELBCKE, F. A. "Surveys for Railway Location," xxix, 429; xxx, 530.

GILLHAM, Robert. "Transmission of Power in Operating Cable Railways," xxix, 543; xxx, 554.

GILPIN, E. Bridge Substructure and Foundations in Nova Scotia, xxx, 573.

GLEIM, C. O.
Improvement of Rivers, xxx, 485.
Railway Terminal Systems, xxx, 512.
Stability of Quay Walls, xxx, 689.
Subaqueous Foundations, xxx, 579.
Surveys for Railway Location, xxx, 526.

GOLDMARK, Henry.
Continuous Superstructure of the Memphis Bridge, xxx, 560.
Terrestrial Magnetism in North America, xxx, 653.

GOODWIN, George A. "The Relative Merits of Working Hoisting Machinery by Steam, Water and Electricity," xxix, 695.

GREINER, J. E. Treatment of Metals for Structural Purposes, xxx, 659.

GRIFFITH, John Purser.
Harbor Improvement, xxx, 490, 498.
Improvement of Rivers, xxx, 496.
Manufacture and Testing of Portland Cement, xxx, 605.
Proposed Method of Testing Structural Steel, xxx, 674.

GUADALAJARA, MEXICO.
"Electric-Light Installation, Utilizing the Famous Juanacatlan Water Falls, 28 Km. Distant from Guadalajara." Rafael M. de Arozarena, xxix, 689.
"Railroads to Connect—with the Pacific Ocean." S. V. Pascal, xxix, 373.

GUILLAIN, F. "Navigation Works Executed in France from 1876 to 1891," xxix, 1.

HAMBURG.
Quay Cranes at— Chr. Nehls, xxx, 258.
Cholera at— W. Kümmel, xxx, 330.

HARBORS.
"Building of Leixões Harbor, Portugal." A. J. N. Soares, xxix, 194.
Commerce of Glasgow Harbor. James Deas, xxix, 158.
French— F. Guillain, xxix, 48.
Port Colborne, Lake Erie—Rock Excavation at. O. M. Poe, xxx, 479.

HARBORS—IMPROVEMENT OF.
Charleston. Wm. M. Black, xxix, 246.
Dublin, Ireland. John P. Griffith, xxx, 491, 499.
Glasgow. James Deas, xxix, 128.
Holyhead, England. John P. Griffith, xxx, 490.
How Provided for. C. W. Raymond, xxx, 464; Wm. M. Black, 500.
Liverpool, England. John P. Griffith, xxx, 500.
"On the South Atlantic Coast of the United States." Wm. M. Black, xxix, 223.
Savannah. Wm. M. Black, xxix, 254.

HARDY, Geo. R.
Railway Signaling, xxx, 546.
Surveys for Railway Location, xxx, 525.

HAZEN, Allen. Purification of Sewage and of Water by Filtration, xxx, 702.

HEIDENREICH, E. Lee. "American Grain Elevators," xxix, 644; xxx, 584.

HERING, Rudolph. Purification of Sewage and of Water by Filtration, xxx, 703.

HERSCHEL, Clemens. Translation of paper by B. Salbach, xxx, 293.

"HISTORICAL NOTES UPON ANCIENT AND MODERN SURVEYing and Surveying Instruments." H. D. Hoskold, xxx, 135.

"HISTORY OF THE CONVERSION OF THE RIVER CLYDE into a Navigable Water-Way, and of the Progress of Glasgow Harbor from its Commencement to the Present Day." James Deas, xxix, 128.

Discussion, xxx: O. M. Poe, 478; James Deas, 482.

HOLLAND.

Amsterdam Ship Canal. A. E. Kempees, xxx, 393; A. Huet, xxx, 416.

North and East Sea Canal. J. Fülscher, xxx, 421.

North Sea Canal of— A. E. Kempees, xxx, 393; A. Huet, xxx, 416.

HOSKOLD, H. D. "Historical Notes upon Ancient and Modern Surveying and Surveying Instruments," xxx, 135.

HUET, A. "Notes on Projects for the North Sea Canal from 1629–1893," xxx, 416.

HUNT, Alfred E.

"A Proposed Method of Testing Structural Steel," xxx, 181, 675.

Treatment of Metals for Structural Purposes, xxx, 657.

HUNT, Charles Warren. Translation of Paper by H. Despres, xxx, 235.

HUTTON, Wm. R.

Improvement of Rivers, xxx, 487.

The North Sea Canal of Holland, xxx, 714.

HYDRAULIC POWER.

For Lifting Purposes. G. A. Goodwin, xxix, 701.

"The Relative Merits of Working Hoisting Machinery by Steam, Water and Electricity. G. A. Goodwin, xxix, 695.

"IMPROVED METHOD OF CONSTRUCTING FOUNDATIONS under Water by Forcing Cement into Loose Sand or Gravel by Means of Air Pressure." Fr. Neukirch, xxix, 639; xxx, 579.

Discussion, xxx: C. O. Gleim, 579; F. Neukirch, 579.

"IMPROVEMENT OF HARBORS ON THE SOUTH ATLANTIC Coast of the United States." William Murray Black, xxix, 223, 500, 509.

Discussion, xxx: John Purser Griffith, 498; Wm. M. Black, 500, 509; Wm. P. Craighill, 503; Thos. W. Symons, 506.

IMPROVEMENT OF RIVERS.

Th. Rehbock, xxx, 483; C. O. Gleim, 485; Wm. M. Black, 486; W. R. Hutton, 487.

"INLAND TRANSPORTATION." F. A. Mahan, xxix, 97.

Discussion, xxx: C. W. Raymond, 457; Edw. P. North, 467; F. A. Mahan, 474.

IRON ORE.

Total Production of—in 1890. C. Weyrich, xxx, 219.

IRON, WROUGHT.
Production of—from 1850 to 1892. C. Weyrich, xxx, 222.

JETTIES.
In Charleston Harbor. Wm. M. Black, 248.
In St. Johns River. Wm. M. Black, xxix, 262, 272.
Proper form for. Thos. W. Symons, xxx, 507.

JORDAN, C. E. Purification of Sewage and of Water by Filtration, xxx, 708.

KECKER, G. "Railway Signaling," xxix, 491.

KEMPEES, A. E.
"The Enlargement and Improvement of the North Sea Canal of Holland (Amsterdam Ship Canal)," xxx, 386.

KING, W. R.
Improvement of Rivers, xxx, 494.
Surveys for Railway Location, xxx, 530.

KINIPPLE, Walter Robert. Manufacture and Testing of Portland Cement, xxx, 604.

KÖPCKE, C., and O. KLETTE. "The New Railway Stations at Dresden," xxx, 450.

KÜMMEL, W. "Some Questions Concerning the Filtration of Water," xxx, 330.

LESLEY, R. W.
Manufacture and Testing of Portland Cement, xxx, 595.
The North Sea Canal of Holland, xxx, 713.

LIFTS. See *Elevators.*

LIGHTHOUSES.
In France. F. Guillain, xxix, 59.

"LIMITS ATTAINABLE IN IMPROVING THE NAVIGABILITY of Rivers by Means of Regulation." H. Engels. (Translated from the German by Kenneth Allen.) xxix, 202.
Discussion, xxx: H. Engels, 492, 497; Wm. M. Black, 493; W. R. King, 494; John Purser Griffith, 496.

LOADINGS FOR RAILWAY BRIDGES.
Ward Baldwin, xxx, 515; J. A. L. Waddell, 522; C. D. Purdon, 523.

LOCKS.
Amsterdam Ship Canal. A. E. Kempees, xxx, 396, 406.
Dimensions of—in France. F. Guillain, xxix, 51.
Fontinette's Elevator, France. F. Guillain, xxix, 17.
Gates for—in France. F. Guillain, xxix, 53.
High Lift— F. Guillain, xxix, 18.
Holtenau—East Sea Canal. J. Fülscher, xxx, 435.
Of Canal St. Denis, France. F. Guillain, xxix, 17.

LOHSE, F. "Rearrangement of Railroad Tracks and Stations in Cologne, Prussia," xxix, 277.

MACDONALD, CHARLES. The Continuous Superstructure of the Memphis Bridge, xxx, 563.

MACHINERY.
Results of Tests of Electric— G. A. Goodwin, xxix, 717.
For Loading and Unloading at Ports of France. H. Despres, xxx, 248.
"Relative Merits of Working Hoisting—by Steam, Water and Electricity." G. A. Goodwin, xxix, 695.

MAGNETISM.
"Terrestrial—in North America." Charles A. Schott, xxx, 108; Discussion, 653.

MAHAN, F. A. "Inland Transportation," xxix, 97; xxx, 474.

"MANUFACTURE AND TESTING OF PORTLAND CEMENT." Henry Faija, xxx, 43.
Discussion, xxx: D. J. Whittemore, 594; R. W. Lesley, 595; Wm. M. Black, 601; Sir Benj. Baker, 603; W. R. Kinipple, 604; John P. Griffith, 605; Henry Faija, 607.

"MANUFACTURE AND USE OF PAVING BRICK." D. W. Mead, xxix, 653.
Discussion, xxx: D. W. Mead, 585, Willard Beahan, 586; F. A. Calkins, 586; Albert Smith, 588.

MAPS, TOPOGRAPHIC.
Value of— Herbert G. Ogden, xxx, 64, 70, 73.
U. S. Coast Survey— Herbert G. Ogden, xxx, 78.

MARITIME COMMERCE.
In France. F. Guillain, xxix, 64.

MARSHALL, H. M. The Use of Long Steel Tapes, xxx, 642.

MATTRESSES.
Comparative value of different— Thos. W. Symons, xxx, 510.
In Cape Fear Dikes. Wm. M. Black, xxix, 245; W. P. Craighill, xxx, 504.
In Charleston Harbor. Wm. M. Black, xxix, 249. 250.
In St. Johns River. Wm. M. Black, xxix, 265.

McCANN, T. H. Translation of paper by L. Franzius, xxix, 173.

MEAD, DANIEL W.
"The Manufacture and Use of Paving Brick," xxix, 653; xxx, 585.
Underground Sources of Water Supply, xxx, 696.

MEASURES.
Fundamental Standards of Length and Mass. T. C. Mendenhall, xxx, 130.

Standard Weights. Origin of— T. C. Mendenhall, xxx, 126.
"Fundamental Units of—" T. C. Mendenhall, xxx, 120.

MEHRTENS, GEORGE C. "The Use of Mild Steel for Engineering Structures," xxx, 204.

"MEMPHIS BRIDGE. The Continuous Superstructure of—" G. S. Morison, xxix, 573; xxx, 564.
Discussion, xxx: 559.

MENDENHALL, T. C. "Fundamental Units of Measure," xxx, 120.

MERRIMAN, MANSFIELD. Translation of paper by F. Lohse, xxix, 277.

METALS.
Treatment of—for Structural Purposes. James Christie, xxx, 155.

METCALF, WILLIAM. The Use of Mild Steel, xxx, 684.

"METHOD USED TO SECURE THE STABILITY OF A QUAY-Wall at the Port of Altona, on the Elbe, in Germany, which had shifted its Position after Completion." Berthold Stahl, xxx, 284.
Discussion, xxx: Peter Vedel, 687; C. O. Gleim, 689.

METRIC SYSTEM.
Perfection of— T. C. Mendenhall, xxx, 125.

MEXICO.
"Railroads in—" E. P. Basave, xxix, 357.

MILLER, R. P. Translation of paper by F. A. Gelbcke, xxix, 429.

MILLS, HIRAM F. "Purification of Sewage and of Water by Filtration," xxx, 350.

MILWAUKEE, WIS.
Sewerage System of— G. H. Benzenberg, xxx, 367.
—River Flushing Works. G. H. Benzenberg, xxx, 367.

MISSISSIPPI RIVER COMMISSION.
Topographic Work of— Herbert G. Ogden, xxx, 74.

MOORE, ROBERT.
Purification of Sewage and of Water by Filtration, xxx, 707.
The Continuous Superstructure of the Memphis Bridge, xxx, 562.
The Use of Long Steel Tapes, xxx, 638.
The Use of Mild Steel, xxx, 683.

MORISON, GEORGE S.
Proposed Method of Testing Structural Steel, xxx, 666.
"The Continuous Superstructure of the Memphis Bridge," xxix, 573; xxx, 564.
The Use of Mild Steel, xxx, 682.

MURPHY, MARTIN. "Bridge Substructure and Foundation in Nova Scotia," xxix, 620; xxx, 576.

"NAVIGATION WORKS EXECUTED IN FRANCE FROM 1876 to 1891." F. Guillian. (Translated from the French by C. L. Crandall and C. W. Sherman.) xxix, 1.

NEHLS, Chr. "The Development of Quay-Cranes in the Port of Hamburg," xxx, 258.

NEUKIRCH, Fr. "Improved Method of Constructing Foundations Under Water by Forcing Cement into Loose Sand or Gravel by Means of Air Pressure," xxix, 639; xxx, 579.

"NEW METHOD OF CALCULATING CROSS-SECTIONS OF Roads and Railroads." Francisco Da Silva Ribeiro. (Translated from the French by Foster Crowell.) xxix, 447.

Discussion, xxx: Arthur Pew, 533; F. Da Silva Ribeiro, xxx, 534.

"NEW RAILWAY STATIONS AT DRESDEN." C. Köpcke and O. Klette, xxx, 450.

NEW SOUTH WALES.

"Railway System of—" T. F. Birrell, xxix, 326.

NEW ZEALAND.

First Railway Line in— T. F. Birrell, xxix, 329.

NICHOLSON, G. B. Surveys for Railway Location, xxx, 526.

NORTH, Edward P. Inland Transportation, xxx, 467.

"NORTH AND EAST SEA CANAL." J. Fülscher. (Translated from the German by Kenneth Allen.) xxx, 421.

"NOTES ON PROJECTS FOR THE NORTH SEA CANAL FROM 1629–1893." A. Huet, xxx, 416.

NOVA SCOTIA.

"Bridge Substructure and Foundations in—" Martin Murphy, xxix, 620; xxx, 576.

OCKERSON, J. A. The Use of Long Steel Tapes, xxx, 639.

OGDEN, Herbert G. "Topographic Surveys," xxx, 62, 632.

"ON THE DISTRIBUTION OF WATER IN THE CITY OF Mexico." L. Salazar, xxx, 336.

"ON THE GAUGES OF RAILROAD TRACK IN GENERAL, with Special Consideration of Narrow-Gauge Railroads." E. A. Ziffer. (Translated from the German by Wolfgang G. Triest.) xxix, 453.

Discussion, xxx: H. G. Prout, 537; Foster Crowell, 538; E. E. R. Tratman, 541; E. A. Ziffer, 541.

ORE.

Total Production of Iron—in 1890. C. Weyrich, xxx, 219.

OTTEWELL, Alfred D. The Continuous Superstructure of the Memphis Bridge, xxx, 565.

OWEN, JAMES. Practical and Æsthetic Principles for the Laying Out of Cities, xxx, 591.

PASCAL, S. V. "Study upon Railroads to Connect Guadalajara with the Pacific Ocean," xxix, 373.

PAVEMENT.
Brick for— D. W. Mead, xxix, 674.
Durability of Brick— D. W. Mead, xxix, 675.

PEW, ARTHUR.
Surveys for Railway Location, xxx, 526.
Calculation of Cross Sections, xxx, 533.

PIERS.
Concrete used for— M. Murphy, xxix, 622.
Of Avon Bridge, N. S. K. Sutherland, xxx, 568.
Of Gaspereaux Bridge, N. S. K. Sutherland, xxx, 568.

PIMENTAL, FREDERICO AUGUSTO. "Common Roads, Railways and River Communications in Portugal," xxix, 299.

PLANE-TABLE.
Method of Using—by U. S. Coast Survey. Herbert G. Ogden, xxx, 76.

"PLANT OF MARITIME COMMERCIAL PORTS OF FRANCE." H. Despres. (Translated from the French by Charles Warren Hunt.) xxx, 235.

POE, O. M. Subaqueous Rock Excavation, xxx, 478.

POOLE, H. S. Bridge Substructure and Foundations in Nova Scotia, xxx, 576.

"PORTS OF FRANCE, PLANT OF MARITIME COMMERCIAL." H. Despres, xxx, 235.

PORTUGAL.
"Common Roads, Railways and River Communications in—" F. A. Pimental, xxix, 299.
Population and Area of— F. A. Pimental, xxix, 300.

POWER.
Transmission of—in Operating Cable Railways. Robt. Gillham, xxix, 543.

"PRACTICAL AND ÆSTHETIC PRINCIPLES FOR THE LAYING Out of Cities." J. Stübben, xxix, 718.
Discussion, xxx: James Owen, 591.

"PROPOSED METHOD OF TESTING STRUCTURAL STEEL." Alfred E. Hunt, xxx, 181.
Discussion, xxx: Geo. S. Morison, 666; L. L. Buck, 667; T. S. White, 668; James Christie, 669; A. C. Cunningham, 669; John P. Griffith, 674; Alfred E. Hunt, 675.

PROUT, H. G. Gauges of Railway Track, xxx, 537.

PURDON, C. D. "Comparison of Modern Engine-Loading, with Standard Specifications for Spans from 10 to 200 Ft.," xxix, 426; xxx, 523.

"PURIFICATION OF SEWAGE AND OF WATER BY FILTRATION." Hiram F. Mills, xxx, 350.
Discussion, xxx: Edw. Flad, 702; Allen Hazen, 702; Rudolph Hering, 703; Thos. M. Drown, 705; G. H. Benzenberg, 706; Robt. Moore, 707; E. C. Jordan, 708.

QUAY WALLS.
At Altona. Berthold Stahl, xxx, 284; Discussion, 687.
At the Harbor of La Plata, Argentine Republic. Peter Vedel, xxx, 688.
At Hamburg. C. O. Gleim, xxx, 689.

RAILROADS.
Avoidance of Grade Crossings at Altona, Germany. R. Caesar, xxix, 295.
First—in Various Countries. E. A. Ziffer, xxix, 457.
First—in Victoria and New Zealand, Australia. T. F. Birrell, xxix, 329.
"In Mexico." E. P. Basave, xxix, 357.
In New South Wales. T. F. Birrell, xxix, 326.
In Portugal. F. A. Pimental, xxix, 299.
Invested Capital of— E. A. Ziffer, xxix, 463.
Location. Geo. R. Hardy, xxx, 525; G. B. Nicholson, 526; C. O. Gleim, 526; Arthur Pew, 526; A. Bryson, 528; W. R. King, 530.
Mileage of—in Europe. T. F. Birrell, xxix, 339.
Mileage of—in the United States. T. F. Birrell, xxix, 339; Foster Crowell, xxx, 540.
Mileage of—in the World. E. A. Ziffer, xxix, 463.
"New Method of Calculating Cross-Sections of Roads and Railroads." F. Da S. Ribeiro, xxix, 447; xxx, 534; Discussion, 533.
"Railroads in the Republic of Mexico in 1893." E. Prieto Basave. (Translated from the Spanish by Foster Crowell.) xxix, 357.
"Railway System of New South Wales." Thomas Fletcher Birrell, xxix, 326.
"Rearrangement of Tracks and Stations in Cologne, Prussia." F. Lohse, xxix, 277.
Regulation of Stations in Prussia, xxix, 286.
Speed on—in England. T. F. Birrell, xxix, 351.
Speed on—in America. T. F. Birrell, xxix, 351.
Speed on—in Australia. T. F. Birrell, xxix, 350.
Terminal System. C. O. Gleim, xxx, 512.

"Surveys for Railway Location." F. A. Gelbcke, xxix, 429; xxx, 530.
"To Connect Guadalajara with the Pacific Ocean." S. V. Pascal, xxix, 373.
Train Sheds—Comparative Size of—in Europe. F. Lohse, xxix, 283.
Train Sheds—Construction of—Cologne, Prussia. F. Lohse, xxix, 284.
Workings. T. F. Birrell, xxix, 355.

RAILWAYS, CABLE.
"Transmission of Power in Operating—" Robt. Gillham, xxix, 543; xxx, 554.
Kansas City— Robt. Gillham, xxix, 544.
Cleveland City— Robt. Gillham, xxix, 553.
Denver City— Robt. Gillham, xxix, 557.
Chicago City— Robt. Gillham, xxix, 565.
Friction Clutches for— Robt. Gillham, xxix, 554.

RAILROADS, ROLLING STOCK.
Cars, Different Types of. T. F. Birrell, xxix, 348.
Cars, Approximate Dead Weight of. D. L. Barnes, xxix, 417.
Cars, Dimensions of, on Narrow-Gauge Railroads. E. A. Ziffer, xxix, 474.
Cars, Lighting of. T. F. Birrill, xxix, 352.
Cars, Statistics of Freight, on Narrow-Gauge Railroads. E. A. Ziffer, xxix, 470.
Freight Trains, Cost for Wages. D. L. Barnes, xxix, 390.
Freight Trains, Weight of. D. L. Barnes, xxix, 388.
Locomotives, American Compound. D. L. Barnes, xxix, 424.
Locomotive Boilers. D. L. Barnes, xxix, 391.
Locomotives, Coal for. D. L. Barnes, xxix, 411.
Locomotives, Construction of. D. L. Barnes, xxix, 390.
Locomotives, Counterbalancing of. D. L. Barnes, xxix, 405.
Locomotives, Driving-Wheel Centers of. D. L. Barnes, xxix, 409.
Locomotives. "Distinctive Features and Advantages of American Locomotive Practice." D. L. Barnes, xxix, 385.
Locomotives, Frames of. D. L. Barnes, xxix, 398.
Locomotives in New South Wales. T. F. Birrell, xxix, 342.
Locomotives, Equalization of Weight of. D. L. Barnes, xxix, 400.
Locomotives, Number in the World. T. F. Birrell, xxix, 348.
Locomotives, Operation of. D. L. Barnes, xxix, 410.
Locomotives, Size of, in America and England. T. F. Birrell, xxix, 342; D. L. Barnes, 424.
Locomotives, Small Drivers for Express Service. D. L. Barnes, xxix, 417.
Locomotives, Tenders of. D. L. Barnes, xxix, 409.
Statistics of— T. F. Birrell, xxix, 339.

"RAILWAY SIGNALING." G. Kecker. (Translated from the German by George F. Swain.) xxix, 491.
Discussion, xxx : Geo. R. Hardy, 546.

RAILROADS, TRACK.
Gauge, Advantages and Disadvantages of Narrow— E. A. Ziffer. xxix, 464, 476.
Gauge, Evolution of— E. A. Ziffer, xxix, 453.
Gauge, Failure of Narrow, in America. Foster Crowell, xxx, 538; E. A. Ziffer, 543.
Gauge, in Different Countries. E. A. Ziffer, xxix, 455.
Gauge, "On the Gauge of Railway Track in General, with Special Consideration of Narrow-Gauge Railroads." E. A. Ziffer, xxix, 453; Discussion, xxx : 537.
Gauge, Specifications for Local Railroads in Germany. E. A. Ziffer, xxix, 487.
Rails, Production of, in 1882. C. Weyrich, xxx, 224.
Rails, Standard for New South Wales. T. F. Birrell, xxix, 333.
Rails, Tensile Strength of Steel for. C. Weyrich, xxx, 224.
"Rearrangement of Railroad Tracks and Stations in Cologne, Prussia." F. Lohse, xxix, 277.
Sleepers, Spacing of, in New South Wales. T. F. Birrell, xxix, 335.
Switches, Protection of Isolated. G. Kecker, xxix, 527.
Switches, Safety. G. Kecker, xxix, 531.
Switch Signals. G. Kecker, xxix, 536.

RAYMOND, C. W. Inland Transportation, xxx, 457.

"REARRANGEMENT OF THE RAILWAY TERMINAL SYSTEM at Altona, with Special Reference to the Avoidance of Grade Crossings." R. Caesar. (Translated from the German by Wm. H. Searles.) xxix, 295. Discussion, C. O. Gleim, xxx, 512.

"REARRANGEMENT OF RAILROAD TRACKS AND STATIONS in Cologne, Prussia." F. Lohse. (Translated from the German by Mansfield Merriman.) xxix, 277.

"RECENT EXPERIENCE ON THE U. S. COAST AND GEODETIC Survey in the Use of Long Steel Tapes for Measuring Base Lines." R. S. Woodward, xxx, 81.
Discussion, xxx: Robt. Moore, 638; Chas. B. Brush, 639; J. A. Ockerson, 639; O. B. Wheeler, 641; H. M. Marshall, 642.

REHBOCK, Theodor.
Improvement of Rivers, xxx, 483.
The North Sea Canal of Holland, xxx, 713.

"RELATIVE MERITS OF WORKING HOISTING MACHINERY by Steam, Water and Electricity." George A. Goodwin, xxix, 695.

RESERVOIRS.
Thermal Storage. G. A. Goodwin, xxix, 698.

RIBEIRO, Francisco Da Silva. "A New Method of Calculating Cross-Sections of Roads and Railroads," xxix, 447; xxx, 533.

RIVERS.
Clyde, Improvement of— James Deas, xxix, 128.
Clyde, Ship-Building on— James Deas, xxix, 163.
Clyde, Navigation on— James Deas, xxix, 168.
Improvement of the Navigability of— H. Engels, xxix, 202; xxx, 492.
Main, Freight Traffic on— Edw. P. North, xxx, 468.
Of France. F. Guillain, xxix, 13.
River Communication in Portugal. F. A. Pimental, xxix, 299.

RIVERS, IMPROVEMENT OF.
By Means of Regulation. H. Engels, xxix, 202; xxx, 492.
Cape Fear, Wm. M. Black, xxix, 236; xxx, 500; W. P. Craighill, 503; T. W. Symons, 508.
Clyde, Scotland. James Deas, xxix, 128; xxx, 482.
In France. Edw. P. North, xxx, 472.
Lower Weser. L. Franzius, xxix, 173; Th. Rehbock, xxx, 483.
Main. Edw. P. North, xxx, 467.
St. Johns. Wm. M. Black, xxix, 262.
Tennessee. Edw. P. North, xxx, 469; W. R. King, 494: H. Engels, 497.
River and Harbor Improvement, How Provided for. C. W. Raymond, xxx, 464; Wm. M. Black, xxx, 500.

ROADS.
In Portugal. F. A. Pimental, xxix, 299.
"A New Method of Calculating Cross-Sections of Roads and Railroads." F. Da S. Ribeiro, xxix, 447.

SALAZAR, L. "On the Distribution of Water in the City of Mexico." xxx, 336.

SALBACH, B. "Experiences Had During the Last 25 Years with Water Works having an Underground Source of Supply," xxx, 293, 701.

SAND.
Effect of Waves on— Wm. M. Black, xxix, 227.
Movements of— Wm. M. Black, xxix, 230, 235, 272; H. Engels, 212; Thos. W. Symons, xxx, 507.
Used in Filtration of Water. H. F. Mills, xxx, 361.

SCHOTT, Charles A. "Terrestrial Magnetism in North America," xxx, 108, 654.

SEARLES, Wm. H.
Translation of paper by R. Caesar, xxix, 295.
Translation of paper by J. Stübben, xxix, 718.

SEWERS AND SEWERAGE.
Analysis of Sewage, at Lawrence, Mass. H. F. Mills, xxx, 356.
Purification of Sewage by Filtration. H. F. Mills, xxx, 350.

"SEWERAGE SYSTEM OF MILWAUKEE AND THE MILwaukee River Flushing Works." G. H. Benzenberg, xxx, 367.
Discussion, xxx: D. J. Whittemore, 709; F. P. Stearns, 710; G. H. Benzenberg, 711.

SIGNALING, RAILWAY.
G. Kecker, xxix, 491; Geo. R. Hardy, xxx, 546.

SHERMAN, C. W. Translation of paper by F. Guillain, xxix, 1.

SMITH, Albert. The Manufacture and Use of Paving Brick, xxx, 588.

SOARES, Alphonso Joaquin Nogueira. "A Brief Account of the Building of Leixões Harbor," xxix, 194.

"SOME QUESTIONS CONCERNING THE FILTRATION OF Water." W. Kümmel, xxx, 330.

SPECIFICATIONS.
For Memphis Bridge. G. S. Morison, xxix, 603.
For Mild Steel Structures. Geo. C. Mehrtens, xxx, 218; C. Weyrich, xxx, 226.
For Paving Brick. D. W. Mead, xxix, 671.
For Portland Cement. Henry Faija, xxx, 60.
For Steel on Baltimore and Ohio Railroad. J. E. Greiner, xxx, 662.

STAHL, Berthold. "Method Used to Secure the Stability of a Quay-Wall at the Port of Altona, on the Elbe, in Germany, which had shifted its Position after Completion," xxx, 284.

STEAM.
"The Relative Merits of Working Hoisting Machinery by Steam, Water and Electricity." G. A. Goodwin, xxix, 695.

STEARNS, F. P. The Sewage System of Milwaukee, xxx, 710.

STEEL.
Annealing— James Christie, xxx, 176.
Apparatus for Testing—by Punching. A. E. Hunt, xxx, 186.
Bessemer—First Use of. Geo. C. Mehrtens, xxx, 204.
Cast—First Use of, for Bridges in America. Geo. C. Mehrtens, xxx, 207.
European Bridges Built of—since 1880. Geo. C. Mehrtens, xxx, 218.
Forging. James Christie, xxx, 175.

Character of Mild Steel. Wm. Metcalf, xxx, 684.
Hardening Tests for— James Christie, xxx, 156.
"Mild Steel for Engineering Structures." Geo. C. Mehrtens, xxx, 204.
Mild—Cheaper than Wrought Iron. C. Weyrich, xxx, 234.
Mild—First Use of—for Bridges. Geo. C. Mehrtens, xxx, 208.
Production of—from 1850 to 1892. C. Weyrich, xxx, 222.
"Proposed Method of Testing Structural Steel." A. E. Hunt, xxx, 181; Discussion, 666.
Riveting. James Christie, xxx, 172; Geo. C. Mehrtens, 217.
Riveted Joints, Experiments on. James Christie, xxx, 167.
Shearing and Punching of— James Christie, xxx, 160.
Shop Treatment of— James Christie, xxx, 159; Geo. C. Mehrtens, 216.
Specifications for Mild—Structures in Several Countries. Geo. C. Mehrtens, xxx, 218; C. Weyrich, 226.
Specifications for—on Baltimore and Ohio Railroad. J. E. Greiner, xxx, 662.
Tensile Strength of—for Structures. James Christie, xxx, 156, 160.
Tension Tests of— James Christie, xxx, 159.
Tests of Metals for Ships. Geo. C. Mehrtens, xxx, 211.
"The Treatment of Metals for Structural Purposes." James Christie, xxx, 155. Discussion, 655.
Total Production of—in 1892. Geo. C. Mehrtens, xxx, 211, 218; C. Weyrich, 220.
"Use of Basic Mild Steel as Material for Construction in Germany." C. Weyrich, xxx, 219. Discussion, 681.

STÜBBEN, J. "Practical and Æsthetic Principles for the Laying Out of Cities," xxix, 718.

"STUDY UPON RAILROADS TO CONNECT GUADALAJARA with the Pacific Ocean." S. V. Pascal. (Translated from the Spanish by Foster Crowell.) xxix, 373.

SUBAQUEOUS ROCK EXCAVATION. O. M. Poe, xxx, 478; James Deas, 482.

SURVEYING INSTRUMENTS.
Ancient and Modern. H. D. Hoskold, xxx, 135.
Use of Long Steel Tapes. R. S. Woodward, xxx, 81; Discussion, 638.

SURVEYS.
Austrian—Cost of. Herbert G. Ogden, xxx, 73.
Cadastral. Herbert G. Ogden, xxx, 64.
Cost of— K. Allen, xxx, 613; H. Gannett, 617.

Cost of—in India. M. Baker, xxx, 619.
Cost of—in the United States. M. Baker, xxx, 623; H. G. Ogden, 633.
Cost of—in Various Countries in Europe. M. Baker, xxx, 620.
"For Railway Location." F. A. Gelbcke. (Translated from the German by R. P. Miller.) xxix, 429; xxx, 530. Discussion, xxx: Geo. R. Hardy, 525; G. B. Nicholson, 526; C. O. Gleim, 526; Arthur Pew, 526; A. Bryson, 528; W. R. King, 530.
German Empire—Cost of. Herbert G. Ogden, xxx, 73.
Holland—Cost of. Herbert G. Ogden, xxx, 73.
India—Cost of. Herbert G. Ogden, xxx, 73.
Ordnance—of Great Britain—Cost of. Herbert G. Ogden, xxx, 73.
Topographic. H. S. Ogden, xxx, 62; Discussion, 611.
Trigonometric—of India. Herbert G. Ogden, xxx, 65.
United States—Cost of. Herbert G. Ogden, xxx, 74.
U. S. Coast and Geodetic. Herbert G. Ogden, xxx, 66.
U. S. Geological. Herbert G. Ogden, xxx, 67.

SUTHERLAND, K. Bridge Substructure and Foundations in Nova Scotia, xxx, 567.

SWAIN, George F. Translation of paper by G. Kecker, xxx, 491.

SYMONS, Thos. W. Improvement of Harbors, xxx, 506.

TAPES, STEEL.
Change in Length Due to Tension. R. S. Woodward, xxx, 104.
Equations of— R. S. Woodward, xxx, 89.
Expansion of— R. S. Woodward, xxx, 90.
Formulas Relative to Length of— R. S. Woodward, xxx, 103.
Use of Long— R. S. Woodward, xxx, 81. Discussion, xxx, 638.

TAYLOR, Edw. B. American Grain Elevators, xxx, 583.

"TERRESTRIAL MAGNETISM IN NORTH AMERICA." Charles A. Schott, xxx, 108.
Discussion, xxx: Henry Goldmark, 653; R. S. Woodward, 653; Charles A. Schott, 654.

"TESTING OF PORTLAND CEMENT AND THE DEVELOPMENT of the Cement Industry in Germany." Max Gary, xxx, 1.
Discussion, xxx: D. J. Whittemore, 594; R. W. Lesley, 595; Wm. M. Black, 601; Benj. Baker, 603; W. R. Kinipple, 604; John P. Griffith, 605.

THERMAL STORAGE RESERVOIRS.
G. A. Goodwin, xxix, 698.

THERMOMETERS.
Sensitiveness of— R. S. Woodward, xxx, 86.

THOMSON, G. H.
The Continuous Superstructure of the Memphis Bridge, xxx, 559.
The Use of Mild Steel, xxx, 681.

TIDES.
In River Weser, Germany. James Deas, xxix, 177.
At Helgoland. James Deas, xxix, 178.
In St. John's River. Wm. M. Black, xxix, 271.

"TOPOGRAPHIC SURVEYS." Herbert G. Ogden, xxx, 62.
Discussion, xxx: Chas. B. Brush, 611; Kenneth Allen, 612; Henry Gannett, 615; Marcus Baker, 619; Herbert G. Ogden, 632.

TOWAGE.
On Canals of France. F. Guillain, xxix, 23.

"TRANSMISSION OF POWER IN OPERATING CABLE RAILways." Robert Gillham, xxix, 543.
Discussion, xxx: A. Bryson, 548; D. Bontecou, 550; Robert Gillham, 554.

TRANSPORTATION.
Freight—on the Great Lakes. F. A. Mahan, xxix, 98.
Freight—in Germany. F. A. Mahan, xxix, 122.
"Inland—" F. A. Mahan, xxix, 97.

TRATMAN, E. E. R. Gauges of Railway Track, xxx, 541.

"TREATMENT OF METALS FOR STRUCTURAL PURPOSES." James Christie, xxx, 155.
Discussion, xxx: L. L. Buck, 655; A. E. Hunt, 657; J. E. Greiner, 659.

TRIBUS, L. L. Underground Sources of Water Supply, xxx, 694.

TRIEST, Wolfgang G. Translation of Paper by E. A. Ziffer, xxix, 453.

TUNNELS.
Severn— H. D. Hoskold, xxx, 154.
Box— H. D. Hoskold, xxx, 153.
Milwaukee Sewage— G. H. Benzenberg, xxx, 367.

"USE OF BASIC MILD STEEL AS MATERIAL FOR CONstruction in Germany." C. Weyrich, xxx, 219.
Discussion, xxx: G. H. Thomson, 681; D. J. Whittemore, 682; Geo. S. Morison, 682; Robt. Moore, 683; Wm. Metcalf, 684; C. Weyrich, 685.

"USE OF MILD STEEL FOR ENGINEERING STRUCTURES." George C. Mehrtens, xxx, 204.

VEDEL, Peter. Stability of Quay Walls, xxx, 687.

VICTORIA, AUSTRALIA.
First Railway Line in— T. F. Birrell, xxix, 329.

WADDELL, J. A. L. Loadings for Railway Bridges, xxx, 522.

WATER.
Average Demand for—in Cities. G. A. Goodwin, xxix, 701.
Bacteriological Examination of— B. Salbach, xxx, 311; H. F. Mills, 352.
Filtration of— B. Salbach, xxx, 294; W. Kümmel, xxx, 330; H. F. Mills, xxx, 350.
Purification of—Containing Iron. B. Salbach, xxx, 296.
Purification of—by Filtration. H. F. Mills, xxx, 350. Discussion, 702.
Purification of—Colored by Turf. B. Salbach, xxx, 297.
Sand Filters for Purification of— W. Kümmel, xxx, 330.

WATER SUPPLY.
Cities in the United States having Subterranean— D. W. Mead, xxx, 698.
Underground Sources of— B. Salbach, xxx, 293; Discussion, 690.

WATER-WAYS.
Of France. F. Guillain, xxix, 4.

WATER WORKS.
Bernburg. B. Salbach, xxx, 302.
Berlin. B. Salbach, xxx, 297; W. Kümmel, xxx, 332.
Cöthen. B. Salbach, xxx, 307.
Crefeld. B. Salbach, xxx, 307.
Dresden. B. Salbach, xxx, 309.
Groningen, Holland. B. Salbach, xxx, 297.
Halle, Germany. B. Salbach, xxx, 294.
Leipzig. B. Salbach, xxx, 301.
Munich. B. Salbach, xxx, 305.
Pressburg. B. Salbach, xxx, 306.
—Having an Underground Source of Supply. B. Salbach, xxx, 293.

WAVES.
Effect of—in Sand. Wm. M. Black, xxix, 227.
Form, Velocity and Energy of— Wm. M. Black, xxix, 225.
Tidal— Wm. M. Black, xxix, 233.

WEIGHTS AND MEASURES.
Standard—Origin of— T. C. Mendenhall, xxx, 126.

WEYRICH, C. "The Use of Basic Mild Steel as Material for Construction in Germany," xxx, 219, 685.

WHEELER, O. B. The Use of Long Steel Tapes, xxx, 641.

WHITE, T. S. Proposed Method of Testing Structural Steel, xxx, 668.

WHITTEMORE, D. J.
Manufacture and Testing of Portland Cement, xxx, 594.
The Sewerage System of Milwaukee, xxx, 709.
The Use of Mild Steel, xxx, 682.
Underground Sources of Water Supply, xxx, 696.

WINDS.
Observations on— Wm. M. Black, xxix, 234.
WOODWARD, R. S.
" Recent Experience on the U. S. Coast and Geodetic Survey in the Use of Long Steel Tapes for Measuring Base Lines," xxx, 81.
Terrestrial Magnetism in North America, xxx, 653.
ZIFFER, E. A. " On the Gauges of Railroad Tracks in General, with Special Consideration of Narrow-Gauge Railroads, xxix, 453; xxx, 541.

NORTHEASTERN UNIVERSITY LIBRARIES
3 9358 00860141 8

NORTHEASTERN UNIVERSITY LIBRARIES
3 9358 00860141 8